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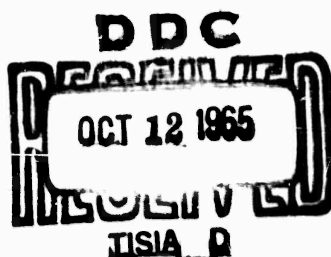
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THE RESPONSE OF SOILS
TO DYNAMIC LOADINGS, REPORT 23

**SEISMIC SHEAR WAVE VELOCITIES
IN SAND AND CLAY**

Frederick V. Lawrence, Jr.



Research Report R65-05
Soils Publication No. 175

January 1965

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Cambridge 39, Massachusetts

Department of Civil Engineering

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THE RESPONSE OF SOILS TO DYNAMIC LOADINGS

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ABSTRACT

Herein is described a study in which the ultrasonic shear wave velocity was measured in sand and clay. The use of the shear wave, rather than the longitudinal wave, offers the possibility of studying stress wave propagation through the skeleton of a soil independently of the pore fluid. The apparatus developed used ferroelectric ceramics to produce and receive a pulsed, torsional shear wave in cylindrical soil samples subjected to various states of hydrostatic stress. Three soil materials were tested: a coarse, rounded sand, and two types of saturated clay.

From the tests on sand it was found that the void ratio and level of effective hydrostatic stress determined the velocity of the shear wave; whereas, the level of shear stress and degree of saturation were found to have only a minor influence.

From the tests involving clay samples, it was found that the effective hydrostatic stress and the void ratio most heavily influenced the shear wave velocity, and the three quantities could be interrelated such that specifying any two would determine the third. Measurements taken during secondary consolidation indicate that the shear wave velocity is sensitive to small structural changes occurring during that period.

PREFACE

Earlier reports, such as Reports No. 14 and 21, have dealt with the measurement of the dilatational wave velocity through soil using the ultrasonic pulse technique. With dry soils, or soils of very low moisture content, the dilatational velocity provides a reasonable measure of the stiffness of the mineral skeleton of the soil. However, for soils having a high degree of saturation, the dilatational wave will usually propagate most rapidly through the pore phase and in such cases the dilatational velocity is not related to the stiffness of the mineral skeleton.

On the other hand, regardless of the degree of saturation, the shear-wave velocity should reflect the stiffness of the mineral skeleton. The present report describes a technique for measuring shear-wave velocity using the ultrasonic-pulse technique, and presents results for this velocity in saturated clays.

This is the twenty-third in a series of reports issued by MIT under the present contract with the U.S. Army Engineer Waterways Experiment Station. A list of the earlier reports follows this preface. This work has been carried out in the Soil Mechanics Division of the Department of Civil Engineering. The work has been supervised by Dr. Robert V. Whitman, Professor of Civil Engineering. The author served as Research Assistant during the period of this study (March 1963-September 1964).

PREVIOUS REPORTS

1. "Scope of Test Program and Equipment Specifications," November 1957.
2. "Test Equipment for High Speed Triaxial Tests," January 1959.
3. "First Interim Report on Dynamic Soil Tests," October 1959.
4. "One-Dimensional Compression and Wave Velocity Tests," August 1960.
5. "Pore Pressure Measurements During Transient Loadings," November 1960.
6. "Effects of Rate-of-Strain on Stress-Strain Behavior of Saturated Soils," April 1961.
7. "Adaptation and Use of the Boynton Device for Rapid One-Dimensional Compression Tests," June 1961.
8. "Laboratory Measurement of Dilatational Wave Propagation Velocity," July 1961.
9. "Shearing Resistance of Sands During Rapid Loadings," May 1962.
10. "Strength of Saturated Fat Clay," June 1962.
11. "Triaxial Tests upon Saturated Fine Silty Sand," December 1962.
12. "Static Tests upon Thin Domes Buried in Sand," December 1962.
13. "The Dependence of Dilation in Sand on Rate of Shear Strain," February 1963.
14. "Propagation Velocity of Ultrasonic Waves through Sand," March 1963.
15. "Undrained Strength of Saturated Clayey Silt," March 1963.
16. "Effective Stress Versus Strength: Saturated Fat Clay," April 1963.
17. "Stress-Strain-Time Behavior of Soil in One-Dimensional Compression," May 1963.

18. "The Dynamic Passive Pressure Problem for Sand," May 1963.
19. "Stresses and Strains in a Planar Array of Elastic Spheres,"
August 1963.
20. "Further Study of a Rapid Response Pore Pressure Gauge,"
November 1963.
21. "One-Dimensional Compression and Wave Propagation,"
October 1963.
22. "Dynamic Response of a Particulate Soil System," March 1964.

TABLE OF CONTENTS

	Page
1. INTRODUCTION	1
1.1 Introduction	1
1.2 Early Studies	1
1.3 Experimental Techniques	2
1.4 Some Results from Wave Velocity Measurements	3
1.5 Considerations Leading to this Study	4
2. EQUIPMENT, PROCEDURES AND MATERIALS USED	6
2.1 Test Chamber	6
2.2 Electronic Equipment	7
2.2.1 Pulse Source	7
2.2.2 Test Chamber	7
2.2.3 Electronic Filter	7
2.2.4 Oscilloscope	7
2.3 Some Characteristics of the Received Signal	8
2.4 Soil Materials and their Preparation	9
2.4.1 Ottawa Sand	9
2.4.2 Boston Blue Clay	9
2.4.3 Kaolinite	10
2.5 Procedures Employed in the Hydrostatic Consolidation Tests	10
3. RESULTS AND OBSERVATIONS	13
3.1 Shear Wave Velocity Variation during Hydrostatic Tests on Sand	13
3.1.1 Tests on Dry Sand	13
3.1.2 Tests on Saturated Sand	14
3.2 Influence of Shearing Stress on the Shear Wave Velocity in Dry Ottawa Sand	14
3.3 Comparisons with Shear Wave Velocities Reported in the Literature	15
3.4 Shear Wave Velocity Measurements during Hydrostatic Consolidation Tests on Saturated Clays	16

	Page
3.4.1 Variation of Velocity with Time during Hydrostatic Consolidation Tests on Clay	16
3.4.2 Variation of Shear Wave Velocity with Effective Stress	18
3.4.3 Variation of Shear Wave Velocity with Void Ratio	19
4. SUMMARY AND FINAL REMARKS	21
4.1 Summary	21
4.2 Final Remarks	22
BIBLIOGRAPHY	23
Plates I - III	
Figures 1 - 23	
<u>Appendix A:</u> Results of X-Ray Diffraction Fabric Studies	
Distribution List	

LIST OF FIGURES

- 1 Elastic Wave Velocities in Ottawa Sand
- 2 Triaxial Cell Converted for Wave Velocity Measurements
- 3 Detail of Triaxial Cell as Modified
- 4 Schematic Representation of Apparatus
- 5 Oscillograph of Received Signal
- 6 Soil Properties
- 7 Summary of Tests Performed
- 8 Results of Hydrostatic Tests on Ottawa Sand
- 9 Influence of Shear Stress on the Shear Wave Velocity in Ottawa Sand
- 10 Results of Shear Test Replotted from Figure 9
- 11 Void Ratio - Pressure Curves for Boston Blue Clay
- 12 Void Ratio - Pressure Curves for Kaolinite Samples
- 13 Time History of Hydrostatic Consolidation Tests on Clay, Load
- 14 Time History of Hydrostatic Consolidation Tests on Clay, Unload
- 15 Results of Hydrostatic Consolidation Tests on Clay
- 16 Results from Loading Portions of Hydrostatic Tests on Boston Blue Clay
- 17 Results from Loading Portions of Hydrostatic Tests on Kaolinite
- 18 Relationship between Void Ratio and Shear Wave Velocity at 20 psi
- 19 Relationship between Void Ratio and Shear Wave Velocity at 40 psi
- 20 Relationship between Void Ratio and Shear Wave Velocity at 70 psi
- 21 Relationship between Void Ratio and Shear Wave Velocity at 100 psi
- 22 Comparison of Void Ratio - Shear Wave Velocity Curves of Various Materials
- 23 Void Ratio - Shear Wave Velocity Behavior during a Typical Test on Clay

CHAPTER 1

INTRODUCTION

1.1 Introduction

The elastic wave velocity of granular materials has recently become a subject of interest to Civil Engineers. Projects involving transient and vibratory loads imposed upon foundations and soil structures are quite frequent, and the need to understand fully their behavior has promoted an exhaustive study of the "dynamic" stress-strain characteristics of soil systems.

Measurements of elastic wave velocities have provided a convenient means of studying the response of soils to rapid loadings. Further, results from such measurements find direct application in situations involving the propagation of seismic waves. Studies of the parameters influencing elastic wave velocities have proven useful in better understanding the role that these "non-dynamic" soil properties play in controlling a soil's behavior.

It is for the last reason that this present study was undertaken. The elastic shear wave velocity* was measured in sands and in clays and an attempt has been made to correlate this with such fundamental soil parameters as intergranular stress, void ratio, soil fabric, and stress history.

1.2 Early Studies

The first studies of elastic waves in granular materials were begun by geologists and seismologists who wished to understand better the behavior of seismic waves in the loose materials comprising the earth's crust. Iida (1939) performed a very thorough study of compression wave and torsional shear wave velocity in granular materials. These studies showed that the velocity of both the compressional wave and shear wave were dependent upon the sixth root sample height (intergranular stress),

* The terms dilatational wave, longitudinal wave, pressure wave, P wave are synonymous and refer to the mechanical wave requiring volume change in the direction of propagation in an elastic space. The terms shear wave, rotational wave, transverse wave, S wave are synonymous and refer to the mechanical wave requiring only rotation and no volume change as the wave propagates through an elastic space. For a dilatational wave particle motion is in the same direction as the progress of the wave; for the shear wave, the particle motion is perpendicular to the wave's motion.

the porosity, and the moisture content. Iida was also the first to show that the dependence of velocity upon the sixth root of intergranular stress could be derived by considering the compliance of two elastic spheres in contact.

The refinements of seismic surveying and the prospect of identifying important mineral strata by the manner in which they transmitted elastic waves were instrumental in renewing interest in this subject, about fifteen years ago. Gassman (1951) and Brandt (1955) advanced the theoretical work started by Iida to include regular packings of like spheres and random mixtures of different spheres. Paterson (1950) showed that unlike a classical solid in which only two independent disturbances could be propagated, the dilatational and shear wave, porous granular materials could support three waves: a dilatational wave through both the mineral skeleton and through the pore fluid, and a third, shear wave, through the mineral skeleton. Biot (1962) set forth a comprehensive theory dealing with all three disturbances. He has pointed out that the two dilatational waves are not entirely independent due to viscous coupling and that the shear wave must also require some rotation of the pore fluid as well as of the mineral skeleton. Although Biot's theory is very comprehensive, application of his study is hindered by the determination of many constants.

1.3 Experimental Techniques

Either of two methods are commonly used to measure elastic wave velocities in the laboratory. The first method is that devised by Ishimoto and Iida (1936). While used in various forms with modifications to suit a particular problem, the method consists of vibrating one end of a cylindrical sample at various frequencies, until the frequency which gives maximum response is found. This frequency is the resonant frequency of the sample. If the mode of vibration is known (i.e., the position of the nodes and antinode along the cylinder), the wave length may be determined from the height of the sample, and the relation,

c = velocity characteristics of the
type of applied vibration, i.e.
shear or compression.

$c = f\lambda$

f = frequency of response

λ = wave length

may be used to calculate the wave velocity. This resonance method has been used by such recent investigators as Shannon, Yamane and Dietrich (1959) and Richart (1962).

The second method consists of initiating a disturbance at one point within a sample and detecting its arrival at a second point. If the separation of the two points is known, the velocity characteristic of the disturbance can be calculated directly from the required transit time. A convenient means of propagating and detecting these disturbances is through the use of piezoelectric crystals or ceramics. Paterson (1956), Lawton (1957), and Lawrence (1963) have employed this technique.

Other than the basic differences in the method used to measure the velocity, the most important distinctions between the two tests are the amount of deformation employed and the frequencies of vibration imposed. In the resonant method, a relatively large deformation (10^{-4} inches) is used to make a measurement; whereas, the pulse technique requires only very small disturbances (about 10^{-6} inches). The frequencies imposed in the pulse type test are generally higher than those necessary to reach resonance in the vibratory test. In the former, the frequencies of the received pulse may be as high as 10,000 c.p.s., while in the latter test, resonance is usually found at frequencies between 200 and 2000 c.p.s.

1.4 Some Results from Wave Velocity Measurements

Comparisons of measured wave velocities with theoretically predicted values have proven interesting. Most investigators have shown that the wave velocity is dependent upon the third to fifth root of intergranular stress, rather than upon the sixth root predicted by theory. This discrepancy has been resolved by Duffy and Mindlin (1957) who showed that even with carefully prepared, regular arrays of extremely high tolerance bearing balls, the relationship between intergranular stress and elastic wave velocity showed the same misbehavior observed earlier in sands; however, as the ambient intergranular stress was increased (beyond about 10 psi), the data began to assume the predicted sixth root dependence. It appears, then, that the wave velocity is dependent not only upon the level of intergranular pressure and void ratio but also upon the number of intergranular contacts existing at any given stress level. In the experiment of Duffy and Mindlin, it seems that the steel balls behaved as expected only after all the possible contacts between particles had been made.

A second question has arisen as a result of measuring elastic wave velocities. If the modulus related to the wave velocity by the equation,

$$c^2 = \frac{M}{\rho}$$

c = elastic wave velocity

ρ = mass density

M = modulus characteristic of the deformation required by the wave's propagation.

was calculated, the resulting value was much higher than any obtained by direct (static) measurement. This discrepancy led to the speculation that for sands the modulus inferred by wave velocity measurements (the "dynamic" modulus) was intrinsically larger than the laboratory measured "static" modulus.

Moore (1963) performed a study relating the "static" and "dynamic" modulus from one-dimensional compression tests to the modulus calculated from measurements of wave velocity. He found that there was no difference between the three if, other things being equal, the stress increment applied to the sand was identical in all three tests.

Richart (1962) adds strength to this assertion by showing the elastic wave velocity to be sensitive to the amplitude of the propagated wave. For low confining pressures (3 psi.), the shear wave velocity was reduced by 10% to 15% by increasing the amplitude of the propagated wave one hundred times (from 10^{-5} radians to 10^{-3} radians). For higher confining pressures (40 psi), the decrease in velocity was much smaller (3%) for the same amplitude increase.

1.5 Considerations Leading to this Study

This present study of shear wave velocities in sand and clay is an extension of previous work by the author in which the dilatational wave speed was measured in two types of dry granular materials, glass beads and Ottawa sand.* In carrying out these previous studies in non-saturated granular systems, measurements of dilatational wave speed by the pulse method proved satisfactory.

Difficulty is encountered, though, in applying this technique to water saturated systems. The dilatational wave borne by the pore fluid is generally of a larger amplitude and of a greater speed than the frame wave. As a result, it is difficult to measure the speed of the frame wave; for it is obscured by the pore wave. Moreover, as Biot (1962) has pointed out, neither the pore wave nor the frame wave of a saturated system is completely independent of the other. The frame wave speed must be influenced by the presence of the pore fluid and cannot reflect solely properties of the mineral skeleton.

For the shear wave, the interaction between the water pore fluid and the mineral skeleton should be quite small; for the distortion characteristic of this wave is a rotation which does

* Lawrence (1963)

not require any volume change of the soil element.* If this interaction is truly small, there should be little difference between the shear wave velocities measured in dry and saturated granular systems. Figure 1 gives a comparison of the elastic wave velocities through dry and saturated Ottawa sand. The dashed lines were taken from Hardin and Richart (1963). The dotted lines were taken from a study by the Shell Development Company.** The solid lines have been obtained by the author. Even though there is slight disagreement as to the actual values of the velocity, there is general agreement as to the extent that saturation influences the shear wave velocity. This seems to be quite small; the average decrease of velocity with saturation seems to be from about 4% to 6%. It appears, then, that in water saturated systems, the shear wave velocity should depend almost entirely upon the mechanical properties of the mineral skeleton, certainly to a much larger degree than the dilatational frame wave. The converse of this finding is that, in saturated systems, measurements of the shear wave velocity provide a means of studying the mechanical properties of the mineral skeleton.

The intent of this present study is to relate the variation of the shear wave velocity and, hence, the mechanical properties of the soil skeleton of a saturated clay, to changes of effective intergranular stress, stress history, void ratio, and particle orientation. To obtain this data, measurements of shear wave velocity were made during the progress of hydrostatic consolidation tests.

* For small vibrations.

** Courtesy of Mr. R. W. Purcell, Director of Basic Research, Shell Development Company, Houston, Texas.

CHAPTER 2

EQUIPMENT, PROCEDURES AND MATERIALS USED

Plate I shows the apparatus of this study in its entirety. At the center of the photograph, are two Genor triaxial cells which were adapted for wave velocity measurements. To the right of these are two air-water accumulators used to supply pressure to the former cells. At the extreme right is the air pressure supply. The electronic equipment necessary for propagating and detecting the pulsed shear wave are grouped at the far left.

2.1 Test Chamber

A Genor triaxial cell converted for shear wave velocity measurements is shown in Plate II and in Figure 2 and 3. The cell accepts a 2 in. diameter sample 3.75 in. in length. The sample is sheathed by filter strips which provide a drainage path down the sides of the sample and down a portion of the pedestal, see Figure 3. At the junction between the Lucite and metal segments of this pedestal is a drainage ring which leads the moisture accumulating in the filter strips outside the cell to a volume measuring burette. A membrane encases the sample and is sealed to the top cap and pedestal by constricting "O" rings.

In the faces of the pedestal and top cap against the sample, twelve piezoelectric elements are mounted in a pattern resembling the spokes of a wheel. Each of these elements are 0.1 in. square and 0.6 in. long. They are composed of a ceramic known as LTZ* and are so manufactured that they will produce or sense shear displacements in the plane of their square section.**

In each face, the twelve elements are connected in parallel; so that if an electric pulse were applied, the surfaces in contact with the sample would move together producing a rapid torsional distortion of that portion of the sample's end. Conversely, if a small torsional displacement were applied to the face of these elements, an electrical voltage could be measured.

* Lead Titanium Zirconate

** Although these elements are primarily shear transducers, they are slightly sensitive to normal pressures and do expand in their thickness dimension when used as a source. This effect is minor compared to the shear displacement sensitivity and was only sufficient to produce a small dilatational wave through the pore fluid of the sample or, more probably, through the water surrounding the sample: see Section 2.3. No dilatational frame waves were observed.

In both the pedestal and top cap, the piezoelectric elements are cemented to a Lucite insulator. The spaces between the elements are filled with latex rubber to present an even surface to the soil sample, and to provide electric insulation.

The sample is loaded by hydrostatic stress from the water pressure in the chamber, or if desired, by a deviator stress from the plunger.

2.2 Electronic Equipment

Figure 4 and Plate III show the instruments used to make velocity measurements. There are four main elements, the pulse source, test chamber, filter, and oscilloscope(s).

2.2.1 Pulse Source

The pulse source generates two signals. The first is a square pulse reoccurring every one-fiftieth of a second with an amplitude of 80 volts and a duration of 6.5 microseconds. This pulse excites the "sending" piezoelectric elements. The second pulse is a lower voltage "spike" which synchronizes the oscilloscope's operation with the main pulse.

2.2.2 Test Chamber

The square pulse from the pulse source excites the piezoelectric elements mounted in the pedestal. In response, the elements produce a high frequency torsional pulse which travels upward through the sample to the receiving elements in the top cap. As the mechanical torsion pulse impinges upon the latter, an electrical signal is produced in proportion to the arriving disturbance. This signal is fed through a filter into the oscilloscope.

2.2.3 Electronic Filter

By the use of this filter various unwanted frequencies in the received signal can be rejected. The function of this instrument will be described in Section 2.3.

2.2.4 Oscilloscope

A Tektronix 535A oscilloscope is used to display the received signal and to make measurements of the time required for the shear wave to travel the sample's length. This particular instrument has a built in delay line which greatly increases the ease and accuracy of time measurements. This device enables one to start the sweep of the oscilloscope at any desired time after the torsion pulse has been initiated in the test chamber. If the sweep is delayed by a time equal to the travel time of the

mechanical disturbance, the signal corresponding to its arrival would then appear at the head of the trace. By reading the amount of delay required to place this signal at the head of the trace, the transit time may be accurately determined.

Delay times so obtained can be read to the nearest microsecond and, generally, any reading can be reproduced to plus or minus one microsecond. According to the manufacturer, the overall error of the oscilloscope and delay line is less than three percent, and barring confusion as to the point at which the wave arrives, it is of the same size as the uncertainty in setting the delay line.

The second oscilloscope shown in Plate III is used to compare the filtered signal to the unfiltered signal as a check for phase shifts that could be introduced by the electronic filter.

2.3 Some Characteristics of the Received Signal

A time history of a received signal is shown in Figure 5 as viewed on the oscilloscope. (In this photograph, time is increasing from left to right.) Three separate features of the signal are distinguishable.

At the beginning of the trace, at "time zero", an upward pulse can be seen. This is due to the small inductive coupling that exists between the sending and receiving transducers in the test chamber.

Though pains were taken with the design of the transducers to minimize such interactions, a small signal is still visible. The latter proves valuable, though, in accurately defining the initiation of the mechanical wave.

At approximately 70 microseconds from time zero, a second feature can be seen. This disturbance has a natural frequency of roughly 500 kilocycles per second. This transit time indicates that for a 3.75 inch sample, the wave travels with a velocity of about 5200 feet per second, the speed of a dilatational wave in water. As previously mentioned, the shear producing piezo-electric elements also produce a small dilatational disturbance which is readily propagated by the highly incompressible pore fluid and chamber water. This signal is thought to reflect qualitatively the character of the sending transducer's mechanical output;* since water, unlike granular materials which behave as

* No direct observation of the sending elements' mechanical output has been possible.

dispersive media, transmits wave forms faithfully. As can be seen in Figure 5, this signal persists for a relatively long time and interferes with the third and most important feature of the trace, the signal resulting from the shear wave's arrival. So that the latter may be clearly viewed, the higher frequency water-borne wave is selectively removed by the electronic filter.

The shear wave signal is seen to begin after a period of 240 microseconds. Transit time measurements were taken from time zero to the beginning of this signal using the delay line in the manner described in Section 2.2.4. The frequency and amplitude of this received signal varied with the intergranular stress prevailing in the soil. Hence, at 100 psi., the frequency is about 50 to 20 kilocycles per second and the amplitude from 20 to 10 millivolts; while at 15 psi., both decreased to about 10 kilocycles per second and 2 millivolts, respectively. Below 10 psi., the amplitude of the signal was generally so small that no measurements could be made.

2.4 Soil Materials and Their Preparation

Three soils were used in this study: a quartz sand, Ottawa sand, and two clays, Boston Blue Clay and Georgia Kaolin. Some properties of these soils are tabulated in Figure 6.

2.4.1 Ottawa Sand

Ottawa sand is a well rounded, pure quartz sand. The sand used in this study was passed through a 20 sieve and retained on a 40 sieve. For tests on saturated samples, the sand to be used was placed in a vacuum, under water, to remove any trapped air bubbles.

2.4.2 Boston Blue Clay

Boston Blue Clay is a silty clay, containing the minerals Illite and Chlorite. As received, it had been dried and ground. To prepare samples with a dispersed* soil fabric, a 3000 cc slurry at 50% water content was prepared, and about 5 gms. of Sodium Tetraphosphate were added. This mixture was evacuated

* An experimental X-ray method of determining soil fabric has been employed in this study and is summarized in Appendix A. The results of this study are in conflict with the anticipated and apparent soil structures. Further study has shown that this X-ray method requires extreme care in sample preparation and that the data are not easily interpreted. For this reason, the results of Appendix A have been discredited and the soil fabrics indicated in the text are believed to be correct.

and then consolidated in a five-inch diameter chamber by steps to 15 psi., whereupon, it was removed and stored in oil until use. Three test samples, 2 inches in diameter and 3.75 inches long could be obtained from the prepared soil.

The flocculated Boston Blue Clay samples were prepared by adding 7 gms. of Sodium Chloride to a 300 cc. slurry and consolidating it as previously described.

2.4.3 Kaolinite

The Georgia Kaolin, Peerless #2, is a very pure Kaolinite clay. The material used in these tests was purchased as a powder, 53% of whose particles were smaller than two microns. The first samples were prepared by mixing the dry clay with tap water to form a slurry with a water content of 190%. The slurry was consolidated and stored in the same manner as the samples of Boston Blue Clay. The resulting samples were only 2.5 inches long. The soil fabric of these samples was flocculated.

Samples having a very highly flocculated soil fabric were prepared by mixing the dry clay with a very pure normal solution of Sodium Chloride to form a 190%w/c slurry. After the clay in suspension had flocculated and settled, the excess salt was removed by leaching it with distilled water. The leached slurry was placed in a ten-inch diameter consolidometer and consolidated to 23 psi. The sample was removed from the consolidometer and stored in oil.

The samples with a dispersed or highly oriented fabric were prepared by adding 1 cc. Sodium Tetraphosphate saturated solution to 600 gms. of the consolidated, salt flocculated sample. The block of clay was placed in a plastic bag, the dispersant added and worked in by hand. The result of this operation was to return the block of soil to the state of a very loose slurry. A test sample was prepared by subjecting this slurry to cycles of heating and evacuation until enough water was removed to yield a highly plastic, but manageable, mass. As a result of this preparation, the sample was not saturated but was filled with pockets of entrapped air.

2.5 Procedures Employed in Setting up Tests

In Figure 7, a summary of tests is given. Five tests were performed on Ottawa sand. Tests 0a, 1-3 used dry Ottawa sand. In test 0b the sand was saturated.

To prepare the dry samples, a split mould lined with a rubber membrane was fitted over the pedestal of the test chamber. Layers of sand were spooned into the mould and each layer was vibrated until a dense condition was reached. When the mould

was full, the top cap was set in place, the membrane secured, and the split mould removed. To prevent the sample from collapsing as the mould was withdrawn, it was confined by a partial vacuum. This vacuum was broken only after the test chamber had been assembled and hydrostatic pressure applied to the sample. At this point, the sample's length was determined by placing the plunger in contact with the top cap and measuring the distance between the top of the plunger and the top of the bushing which guides it: see Figure 2. Knowing this distance, the actual sample length could be calculated. Since the length is needed to calculate only the shear wave velocity, it was necessary to measure it only once; for it changed very little during tests. For the hydrostatic tests, the pressure was increased in steps, and the shear wave's transit time was determined at each pressure level.

Unlike the rest of the tests on dry sand, Test 1 involved the application of shearing stresses. A conventional triaxial loading frame was used to supply the desired deviator force, the level of which was read from a proving ring. Changes in sample length were measured with an extensiometer.

For the test on saturated sand, the procedures employed differed only in the following respects. The sand was placed in the mould under water to avoid introducing air. A much smaller vacuum was placed on the sample to avoid cavitation and, hence, partial saturation. Using these reduced vacua proved tricky, for the weight of the top cap was often enough to buckle or deform the sample.

As listed in Figure 7, twelve tests were performed on clay samples. Tests 5, 6 and 7 used dispersed Boston Blue Clay; tests 8 and 9, flocculated. Tests 10 and 13 were performed with Kaolinite samples prepared with tap water. Tests 11, 12, 14 and 15 involved the salt flocculated Kaolinite. Kaolinite remoulded with Sodium Tetraphosphate was used in 16.

The clay material, having been previously consolidated, was trimmed to sample size and mounted on the pedestal. Filter strips and the thick rubber membrane were carefully placed around the sample and smoothed to expel any trapped air. A small amount of water was poured in at the top of the sample to help displace any air pockets. The top cap was installed, and the chamber assembled. A burette was connected to the drainage outlet to measure the volume of water expelled during the course of consolidation. The sample length was calculated in the same manner as before. The previous consolidation pressure was then re-applied, (15 psi for Boston Blue Clay and 23 psi for Kaolinite), and the sample was allowed to consolidate under this stress for at least 1000 minutes. Measurements of transit time were taken intermittently, and the sample's volume change noted. Before the hydrostatic pressure was increased for the next increment, the length, final volume change, and transit time were measured.

In this manner, the samples were loaded incrementally to 100 psi.; whereupon, the unloading sequence commenced or the test was discontinued.

Water content determinations were made both at the start and finish of the test. Calculations of void ratio were generally made from the final water content because of the uncertainty in the water volume measurement of the first load increment due to water introduced between the membrane during preparation.

CHAPTER 3

RESULTS AND OBSERVATIONS

3.1 Shear Wave Velocity Variation During Hydrostatic Tests on Sand

Figure 8 shows the result of five tests in which increasing levels of hydrostatic pressure were applied to Ottawa sand samples, and the shear wave velocity measured. In this plot, the dots represent a value of velocity measured in dry sand; the squares indicate values for saturated sand. The uppermost line bounds the highest measured values of shear wave velocity in dry sand for the pressure range of 10 to 100 psi. The lower line connects the data for saturated sand.*

3.1.1 Tests on Dry Sand

The data shown as dots in Figure 8 are for tests 0a, 1, 2 and 3 on dry Ottawa sand. Since the data fall within a very narrow range of values, no attempt has been made to indicate that some points are for the initial loading while others were obtained during subsequent unloadings and reloadings; nor has any distinction been made between the various tests. At 100 psi, the highest value of velocity is 1610 ft/sec.; the lowest is 1530 ft/sec., or 5% lower. At 10 psi, the range is greater. The highest measured value at this pressure is 930 ft/sec., the lowest is 800 ft/sec., or 14% lower. This variation can be attributed to slight differences in void ratio between tests: the densest samples ($e = 0.54$) yielded the highest values; the less dense samples ($e = 0.59$) yielded the lowest.** The slope of the upper envelope which reflects the behavior of the more dense samples is about 1:4.0. The least dense sample, that of Test 2, showed the greatest measured slope, 1:3.0, in the 10 to 30 psi. pressure range. In the pressure range of 40 to 100 psi., the slopes of all the dry tests were the same as that of the upper envelope, 1:4.0.

* The log-log presentation of the hydrostatic pressure-wave velocity data is customary and is used throughout this study. Presenting the results in this manner, offers two advantages. Since the wave velocity varies with some root of the intergranular stress, (i.e., fourth through sixth), the data will plot as a straight or a very slightly curved line; the slope of which immediately furnishes the apparent root of intergranular stress upon which the wave velocity depends.

** The void ratio was not consistently determined in these tests. In all tests, an effort was made to produce dense samples (i.e., $e = 0.52 - 0.55$) by vibrating and tapping the sides

3.1.2 Tests on Saturated Sand

Also shown in Figure 8 are the results of Test 0b on saturated Ottawa sand. The measured shear wave velocities shown in squares lie slightly lower than the value for dry sand, particularly in the pressure range of 10 to 30 psi. In the higher pressure ranges, 40 to 100 psi., the data are well within the range of values obtained with dry sand. The void ratio of this test is not accurately known. The slope of 1:3.7 and the slight curvature of the data suggest that the sample might be relatively loose, perhaps with a void ratio of approximately 0.60.

3.2 Influence of Shearing Stress on the Shear Wave Velocity in Dry Ottawa Sand

The results of Test 1 in which triaxial shearing stresses were applied to a dry sand sample confined at various hydrostatic stresses, are shown in Figure 9. Here the level of maximum shearing stress is plotted against the average hydrostatic stress (or average principal stress). The lines connecting the points indicate the loading path taken. The loading path originating at 40 psi. was taken to failure, and the Mohr-Coulomb failure surface was sketched. The shear wave velocity was measured at regular intervals, and, in Figure 9, is found adjacent to its appropriate stress. Vertical contours of equal shear wave velocity could be drawn for the velocities of 1260 ft/sec., 1410 ft/sec. and 1490 ft/sec. This fact suggests that the shear wave velocity depends only upon the hydrostatic stress and is very little influenced by the level of shear stress. As a result of this latter observation, the data was plotted in Figure 10 as a function of hydrostatic stress, alone.

In Figure 10, the large points designate the shear wave velocity at the start of each of the loading paths shown in the previous figure, Figure 9. At these points the stress is purely hydrostatic. The smaller points mark the shear wave velocity for the increasing level of hydrostatic stress due to the addition of deviator stress (note the direction of the arrows in Figure 10). It can be seen that all of these points, large and small, lie very close to the upper envelope for dry Ottawa sand redrawn here from Figure 8. Figure 10 gives strong

of the mould. In Test 2, a void ratio of 0.59 was computed. This test supplied the smallest measured wave velocity. Estimated void ratios are given in the text and in Figure 7. In general, a very small range of void ratios is possible with this material. Extreme values of 0.50 and 0.69 have been reported for 20 - 40 Ottawa sand with 0.53 - 0.57 being typical for moderately dense samples.

evidence that it is the hydrostatic component of stress which controls the velocity of shear waves in sand.*

One curious feature of the data in Figure 10 is the small undulations associated with the increasing levels of shear stress. These are thought to reflect changes in void ratio produced by the action of shearing strain. If this is so, and if it can be assumed that the small points would lie on a straight line parallel to the upper envelope were there no void ratio change, then these undulations must reflect the void ratio changes produced during the application of shear stress. Recalling the effects of void ratio on velocity discussed in the previous section and following the loading which starts at 40 psi., the void ratio seems to decrease slightly and then increase considerably at failure (marked with a cross). At 80 and 94 psi., the void ratio seems to decrease abruptly with the addition of shear stress and then gradually increase.

3.3 Comparisons with Shear Wave Velocities Reported in the Literature

Turning back to Figure 1 and confining our attention to the plots of shear wave velocity, it can be seen that there is good agreement as to the slope of the data but some uncertainty as to value. The shear wave velocities presented in Section 3.1 for dry Ottawa sand surpass those reported by Shell Development Company which in turn are greater than any reported by Richart (1963). Since all tests shown here are for dense 20 - 40 Ottawa sand, ** there should be no difference in the properties of the material being studied. One plausible explanation is that in both the Shell tests and those reported by Richart, higher energy shear waves have been used. If this could be proven, the arguments advanced earlier in Section 1.4, that the shear wave velocity should decrease with increasing wave amplitude, would serve to reconcile these apparent differences. As mentioned in Section 1.3, the resonance method used by Hardin and Richart and others employs relatively large vibrations. The pulse method used by Shell and the present study relies upon much smaller disturbances. It is suggested, then, that

* Shannon, Yamane and Dietrich (1960) present data from triaxial tests in which the elastic modulus was determined from resonance measurements of the "compression" wave velocities in (laterally unrestrained) cylinders of sand. These data have been tentatively correlated with σ_1 , the major principal stress, rather than with σ_2 , the minor principal stress.

** Richart used 20 - 30 Ottawa sand.

the closer agreement (7% at 60 psi.) between the present study and that of the Shell Development Company is due to the similarity of propagated disturbances; while the discrepancy (16% at 60 psi.) with Richart's data may be attributed to his use of larger wave amplitudes.

As previously noted, the results of the three studies agree as to the effect of saturation on the shear wave velocity in Ottawa sand. Shell Development Company, Hardin and Richart, and the present study show the shear wave velocity to decrease about the same amount with saturation; however, the same apparent discrepancies in absolute value exist.

3.4 Shear Wave Velocity Measurements During Hydrostatic Consolidation Tests on Saturated Clays

Figures 11 and 12 show the relationship between effective hydrostatic consolidation stress and void ratio for the eleven tests to be presented. Figure 11 shows the behavior of the flocculated and dispersed Boston Blue Clay samples. The dispersed samples are seen to give slightly higher void ratios than do the flocculated samples. Figure 12 shows the behavior of the three Kaolinite soils tested. The remoulded and dispersed sample is seen to have had the lowest void ratios and compression index. The tap water samples were intermediate between the former and the salt flocculated samples which yielded the highest void ratios and compression indices.

3.4.1 Variation of Velocity with Time During Hydrostatic Consolidation Tests on Clay

Figures 13 and 14 illustrate the variation of volume change and shear wave velocity during successive increments (and decrements) of a typical hydrostatic consolidation test on clay.* In these figures, the horizontal scale is the duration of the stress increment (log. time in minutes). Each heavy vertical line marks the beginning or end of a stress increment, the value of which is given at the top of the figure in psi. Three separate curves are shown. Two relate to volume strains occurring in the sample. The third gives the value of shear wave velocity.

The volumetric strain of the sample, calculated from the water squeezed out of the sample, is shown plotted as squares. The volumetric strain inferred by measurements of axial strain at the end of each increment are plotted as triangles. In most

* For the sake of consistency, all "typical behavior" data shown in this study are for one test, No. 15, on saturated, salt-flocculated Kaolinite.

tests, the two do not agree very well during the first few increments; but, there is reasonable agreement during the ensuing increments. One explanation is that the samples were prepared by one-dimensional consolidation and are therefore anisotropic. Until the effects of this previous stress history are removed, the volume change inferred by the axial strain,

$$\frac{3\Delta S}{S_0} = \text{volume strain} = \frac{\Delta V}{V_0}$$

where, $\frac{\Delta S}{S_0}$ is the axial strain, and

$\frac{\Delta V}{V_0}$ is the volumetric strain.

must be different from the true volumetric strain. Then, too, the volumetric strain calculated from the burette readings are suspect during the first increments because of the unknown quantity of water introduced during the sample's preparation. The two really cannot be reconciled at the present time. Fortunately, the discrepancy is not crucial in the arguments to follow. It should be noted that both volume strain measurements, while plotted as positive quantities, are in fact negative, that is, volume decreases. This has been done to facilitate comparisons with the third curve in Figure 12 and 13, the shear wave velocity.

The shear wave velocity, plotted in circles can be seen to vary in the same manner as the volumetric strain $\left(\frac{\Delta V}{V_0}\right)$.

Throughout all tests on clay, this pattern of behavior was very consistent. This seems to suggest that, as with sand, the value of shear wave velocity is dependent upon the level of effective (intergranular) stress.

Three curious features often observed are pointed out by arrows in Figures 13 and 14. The first is shown in the 70 psi. increment of Figure 13. Here, the shear wave velocity at 10 minutes into the 70 psi. increment is lower than the final value for 50 psi. This phenomenon was often noticed during the start of a loading increment preceeded by an increment allowed to start for an exceptionally long period (here: 9000 minutes or six days). The cause of this drop may involve the mechanisms of secondary consolidation. One speculation is that during the extended secondary consolidation period, the clay particles, under the action of the intergranular stress, slowly approach each other more closely as the last few layers of water molecules are expelled from the areas of mineral contact. If this were happening, it is possible that the rigidity of the clay fabric

would increase, thus, increasing the shear wave velocity. When a higher stress increment was started and micro-shear applied to the soil fabric, the motion required of the clay particles to adjust to the load would interrupt these close contacts. This would result in a loss of rigidity and, hence, shear wave velocity.

A second, perhaps related, phenomenon is shown in Figure 14 at the end of the 40 psi. decrement, well into the secondary consolidation period. Here again, the shear wave velocity shows an inexplicable increase while the volume strain (hence void ratio) remains constant. The same thoughts concerning the previous effect may apply here.*

The third phenomenon is shown in Figure 14 at the beginning of the 70 psi. decrement. Here, the shear wave velocity is seen increasing at the beginning of a decrement. The cause of this anomaly is the necessity at 100 psi. of dropping the cell pressure to zero in order to measure the sample's length. Although the stopcock to the burette is closed during this operation to prevent rebound, either partial saturation or water cavitation in the pores of the soil allow a decrease in the effective stress, and, therefore, shear wave velocity. When the pressure is replaced and later dropped to 70 psi., the shear wave velocity is still seen to be recovering.

3.4.2 Variation of Shear Wave Velocity with Effective Stress

Figure 15 shows the relationship between the effective hydrostatic consolidation stress and the final shear wave velocity of that increment. The results in this figure are typical of the behavior observed in all tests with clay. The initial loading curve is approximately a straight line and of lower velocity than the more curved rebound and reload curves.

Figure 16 shows the same type of plot for the loading portions of the four tests on Boston Blue Clay. Here the data form straight lines, with the velocities of the dispersed Boston Blue Clay samples having characteristically higher values than the flocculated. The slope of the lines is about 1:2.5.

Figure 17 shows results for the seven tests on Kaolinite samples. Again only the loading portion is shown and, to standardize the amount of secondary consolidation included, the velocities shown are the values measured at 1000 minutes. Three straight lines result. The bottom is for the salt-flocculated samples. The middle corresponds to the samples prepared with tap water. The highest values are for remoulded samples prepared with a dispersant. The slopes of the three are 1:2.9, 1:2.9 and 1:3.7, respectively.

* This behavior can also be seen in the 20 psi. and 10 psi. decrement of Figure 14. It also appears in the unloading portions of Tests 10, 11 and 14 (not shown).

From the data of Figures 16 and 17, it appears that the relationship between the log of hydrostatic consolidation stress and the log of shear wave velocity is a straight line for the first loading; moreover, the values of velocity for any one clay with a particular fabric tend to fall on one line. As yet, no particular significance can be attached to the slopes of these lines.

3.4.3 Variation of Shear Wave Velocity with Void Ratio

As seen in Figure 15, the velocities measured during the initial loading are lower than those in the subsequent rebound and reload. This behavior was noted in all tests with clay and suggests that some parameters other than effective hydrostatic stress heavily influence the shear wave velocity. The differences between the velocities measured in the two varieties of Boston Blue Clay and the three varieties of Kaolinite indicate that the void ratio, particle orientation, or both may be the sought variables: see Figures 16 and 17, also Figures 11 and 12. Since void ratio was seen to play an important role in sands, it was examined first.

To study the effects on shear wave velocity, the latter was plotted against void ratio in the manner suggested by Iida (1939) and Richart (1963) in their studies on sand. In Figures 18 through 21 the void ratio and the related shear wave velocity for all the clay samples tested are plotted for several levels of effective consolidation stress. The unshaded data points were obtained during the initial loading; the shaded points are from either the unload or reload portions of a test. For a given stress, the data fall along a broad band through which a single straight line has been drawn. These lines have been traced onto Figure 22 where the apparent interrelationship of shear wave velocity, void ratio, and effective consolidation stress is given (solid lines). In Figure 23, the shear wave velocity-void ratio history of a typical test has been plotted. The values of hydrostatic stress are written next to each data point, and the lines of Figure 22 have been superposed in this plot to allow comparisons. While there is some scatter in the data, the tendency of the void ratio-velocity data throughout the test to fall along the superposed lines is quite apparent.

It seems that the void ratio is an extremely important variable in determining the level of shear wave velocity. In fact, from the results presented in Figures 18 to 23, it appears that stress history, mineral type, grain size and particle orientation are of minor importance and influence the shear wave velocity only in as much as they affect the void ratio. Further, it appears that the shear wave velocity, void ratio and effective consolidation stress are uniquely related such that specifying any two will determine the third. For example, if the shear wave velocity and the void ratio of a clay were known, the effective consolidation stress could be predicted from established plots similar to Figure 22. This relationship should also hold at any time during the course of consolidation, since all three

quantities change simultaneously and at the same rate.*

An interesting comparison is given in Figure 22 between the results obtained for two types of clay and the data published by Richart (1963) for dry, crushed, quartz sand (dashed lines). The slopes of the two sets of curves are remarkably similar, and the shear wave velocities only differ by about 15%.

In Figure 22, the circles designate the shear wave velocity-void ratio data for Ottawa sand.

* In Section 3.4.1, the shear wave velocity was shown to change proportionately to the volumetric strain during consolidation. The change in volumetric strain is usually assumed to be proportional to the change in pore pressure which in turn is related to both the change in void ratio and effective stress.

CHAPTER 4

SUMMARY AND FINAL REMARKS

4.1 Summary

From the measurements of shear wave velocity in Ottawa sand the following observations were made:

- (a) The shear wave velocity in dense Ottawa sand varies with the fourth root of intergranular stress, i.e., varies uniformly with hydrostatic stress.
- (b) The level of shear wave velocity is sensitive to small differences in void ratio.
- (c) Applied shear stresses influence the shear wave velocity only in as much as they contribute to the hydrostatic component of stress or produce void ratio changes.
- (d) The decrease of shear wave velocity with saturation of the sand is very slight, and is more pronounced in the lower pressure ranges of 10 to 30 psi.

From the hydrostatic consolidation tests on two clays in which shear wave velocities were measured, the following was concluded:

- (a) During the progress of consolidation, the shear wave velocity varied in the same manner as the volumetric strain and, hence, the dissipation of pore pressure.
- (b) The shear wave velocity seems to be slightly sensitive to changes that occur during secondary consolidation.
- (c) For a particular clay, the relationship between the log. hydrostatic consolidation stress and the log. shear wave velocity is a straight line during the initial loading.
- (d) The effective stress and void ratio are the parameters which most heavily influence the shear wave velocity, and the three are uniquely related such that specifying any two will determine the third.

4.2 Final Remarks

At the present time, the factors influencing the shear wave velocity in sands are quite well understood. The little work remaining to be done here is to adequately explain the discrepancies in measured values of velocity.

For clays, the importance of the void ratio and effective stress in determining the speed of the shear wave was shown in this study. More work is needed to understand the role that soil fabric, mineralogical composition, soil particle geometry, wave amplitude and frequency play in determining the shear wave velocity.

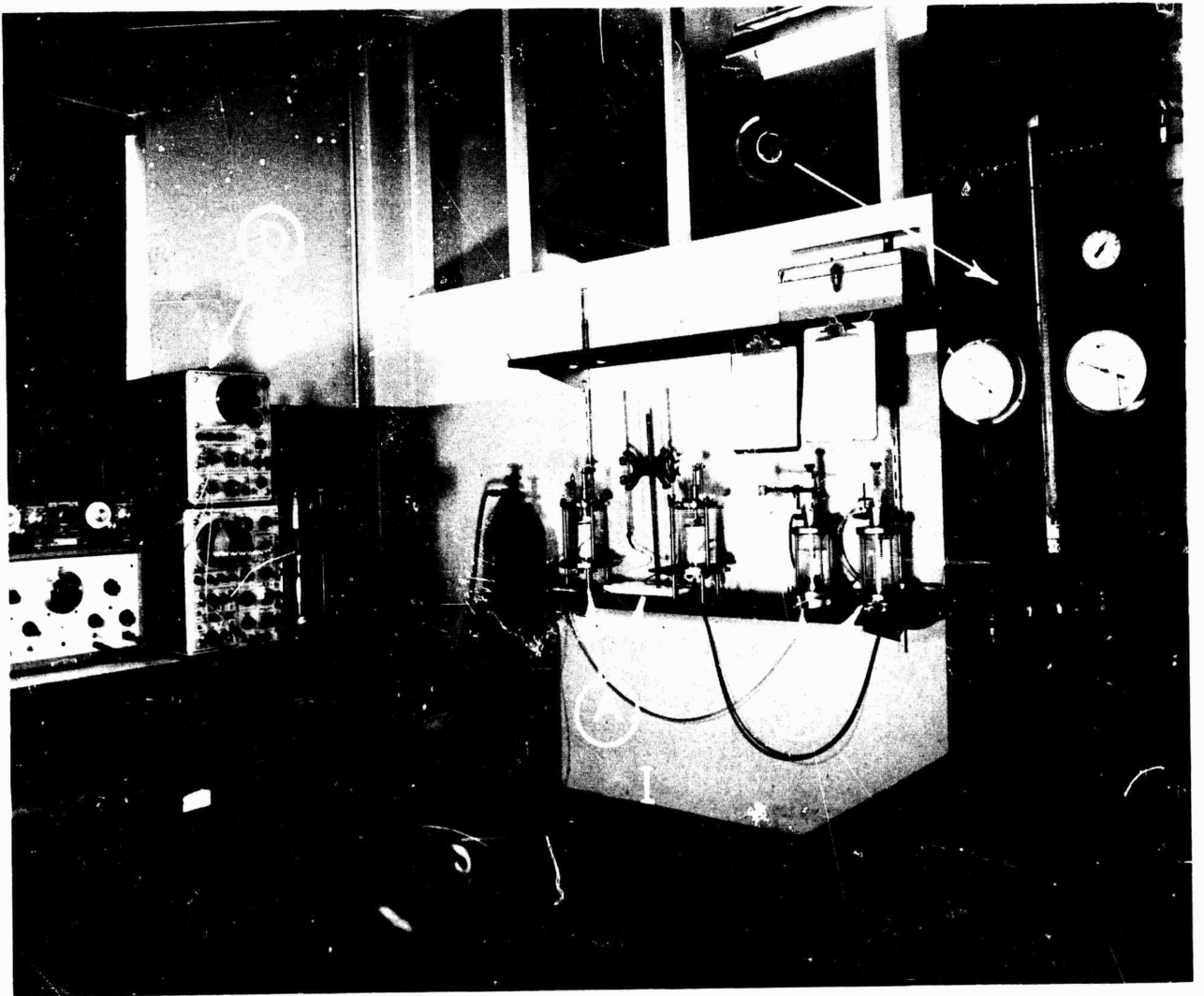
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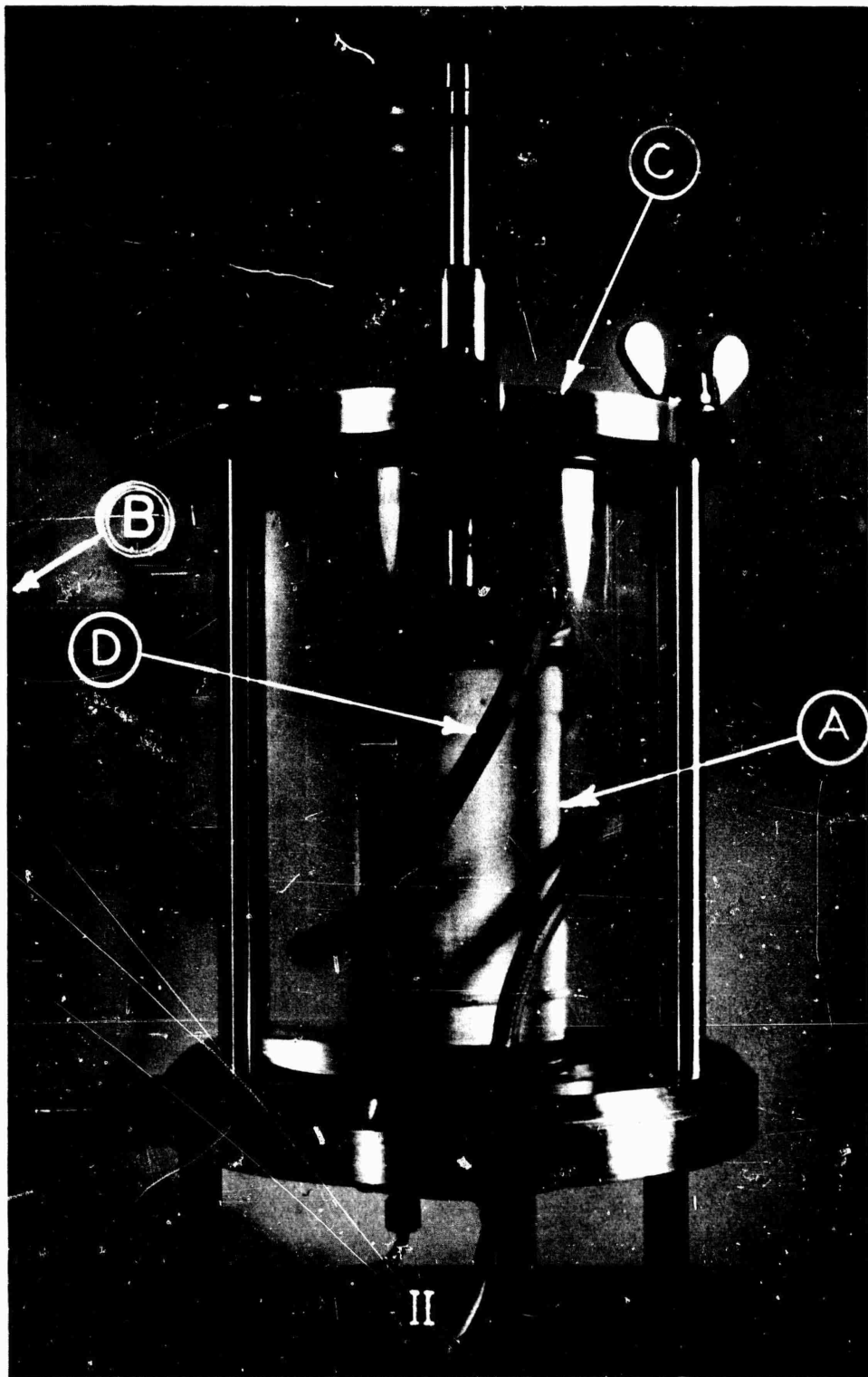
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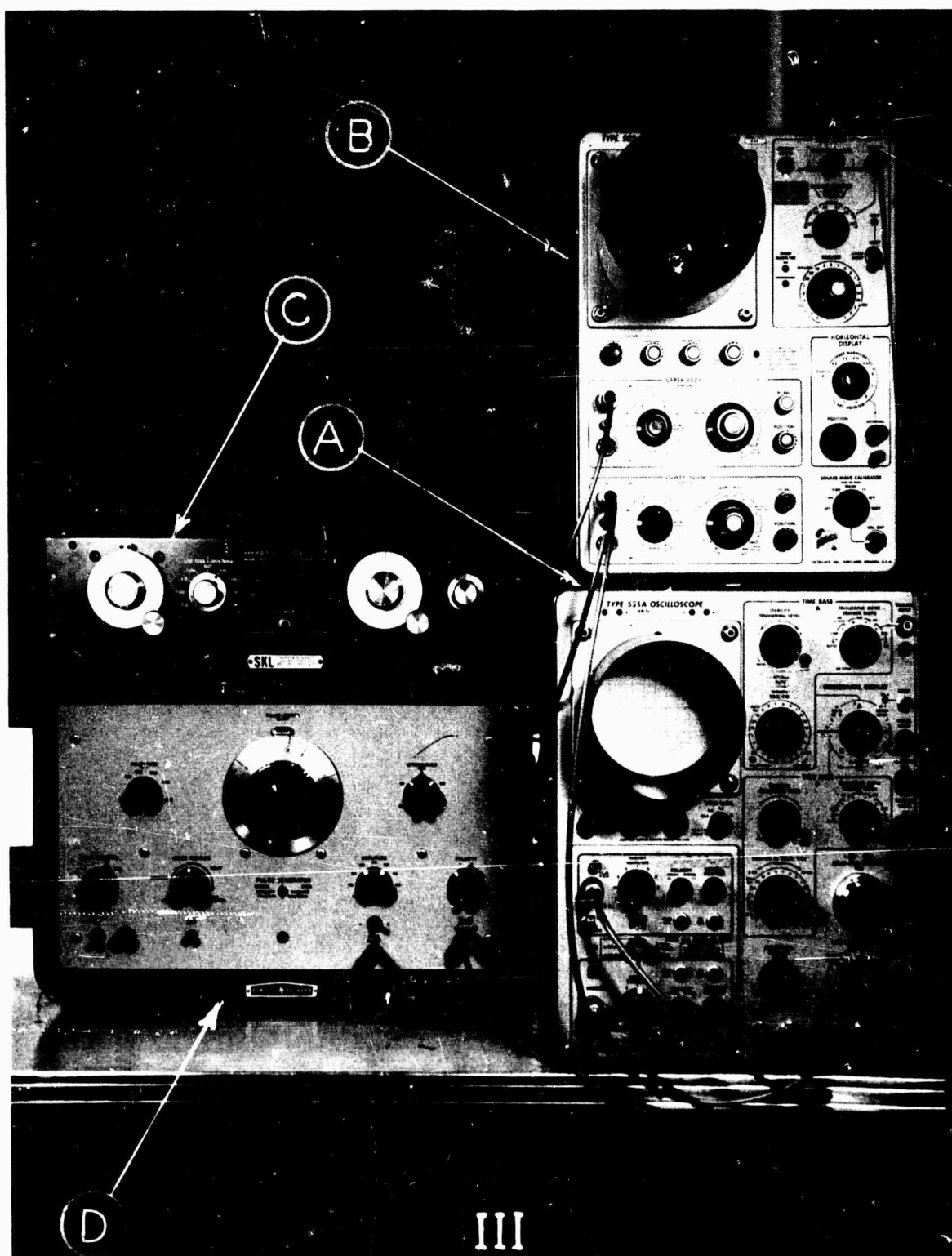
- A....Triaxial cells modified for wave velocity measurements
B....Air-water accumulators
C....Air pressure supply
D....Electronic equipment

PLATE I



- A....Sample enclosed in rubber membrane
B....Volume measuring burette
C....Uniaxial cell, proper
D....Tygon tubing containing wire for received signal

PLATE II



- A....Tektronix 535A oscilloscope
 B....Auxilliary Tektronix 502 oscilloscope
 C....SKL 302 variable electronic filter
 D....Hewlett Packard 212A pulse generator

PLATE III

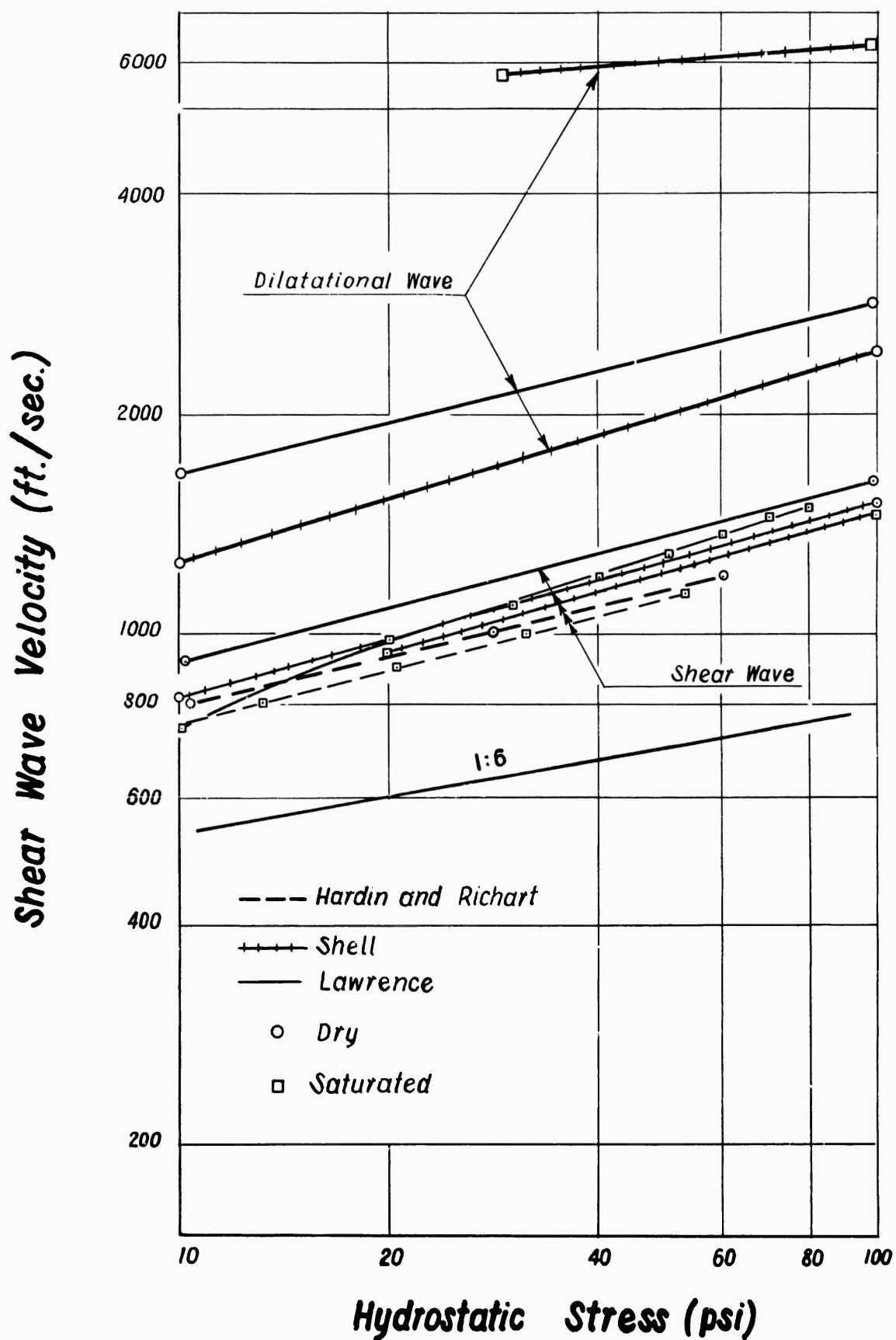


Fig. 1 Elastic Wave Velocities in Ottawa Sand

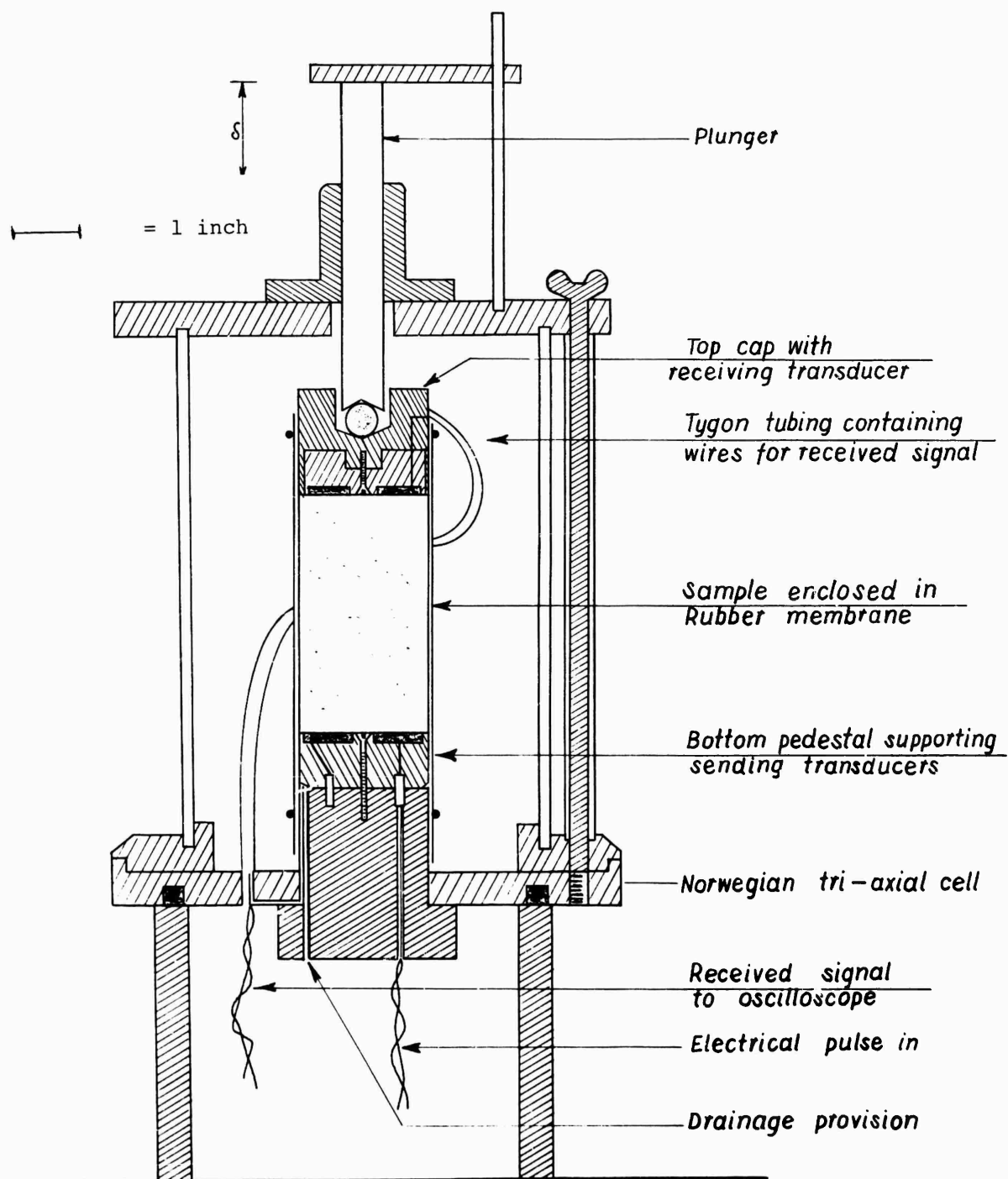


Fig. 2 Tri - axial Cell Converted for Wave Velocity Measurements

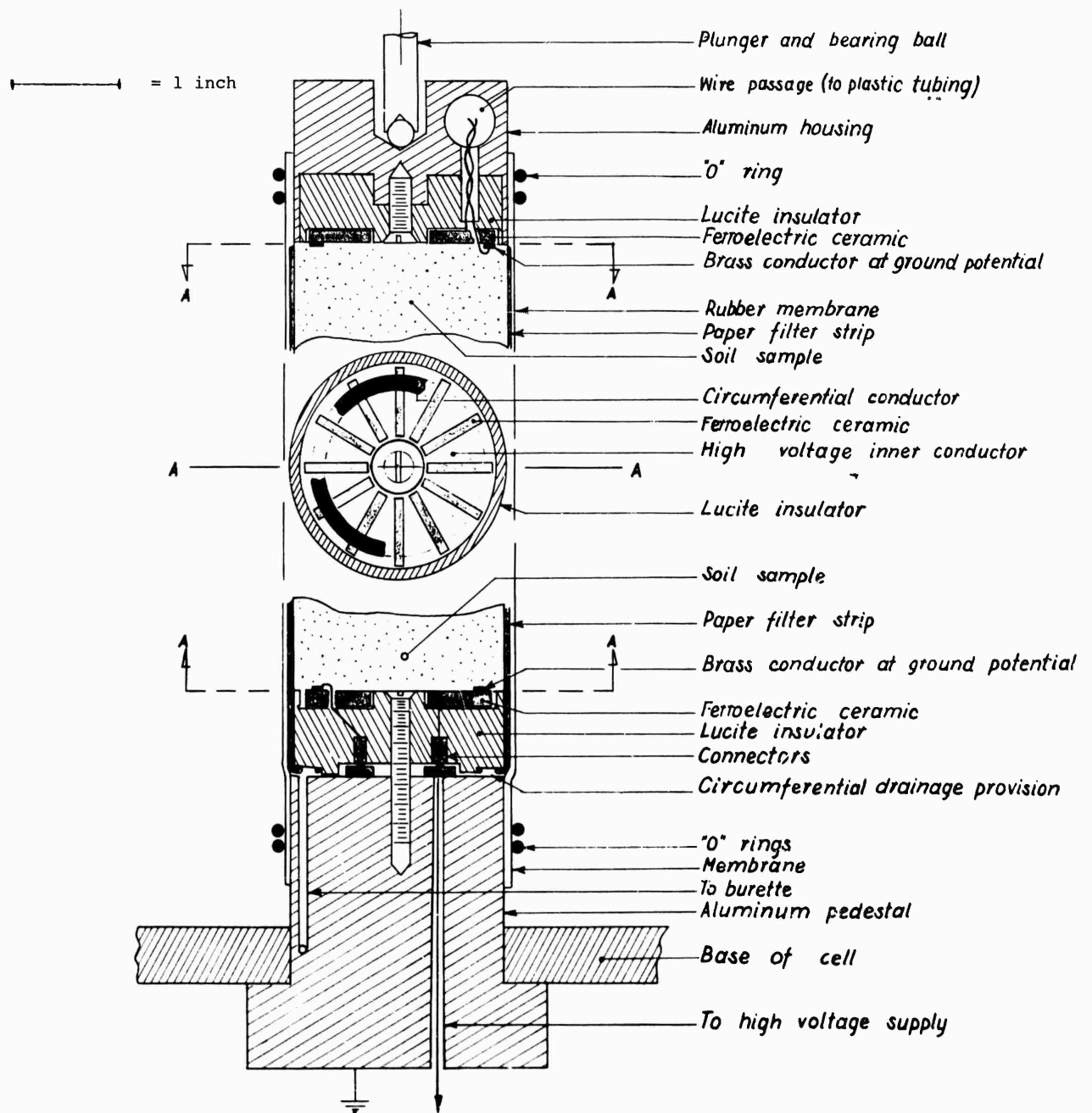


Fig.3 Detail of Tri-axial Cell as Modified

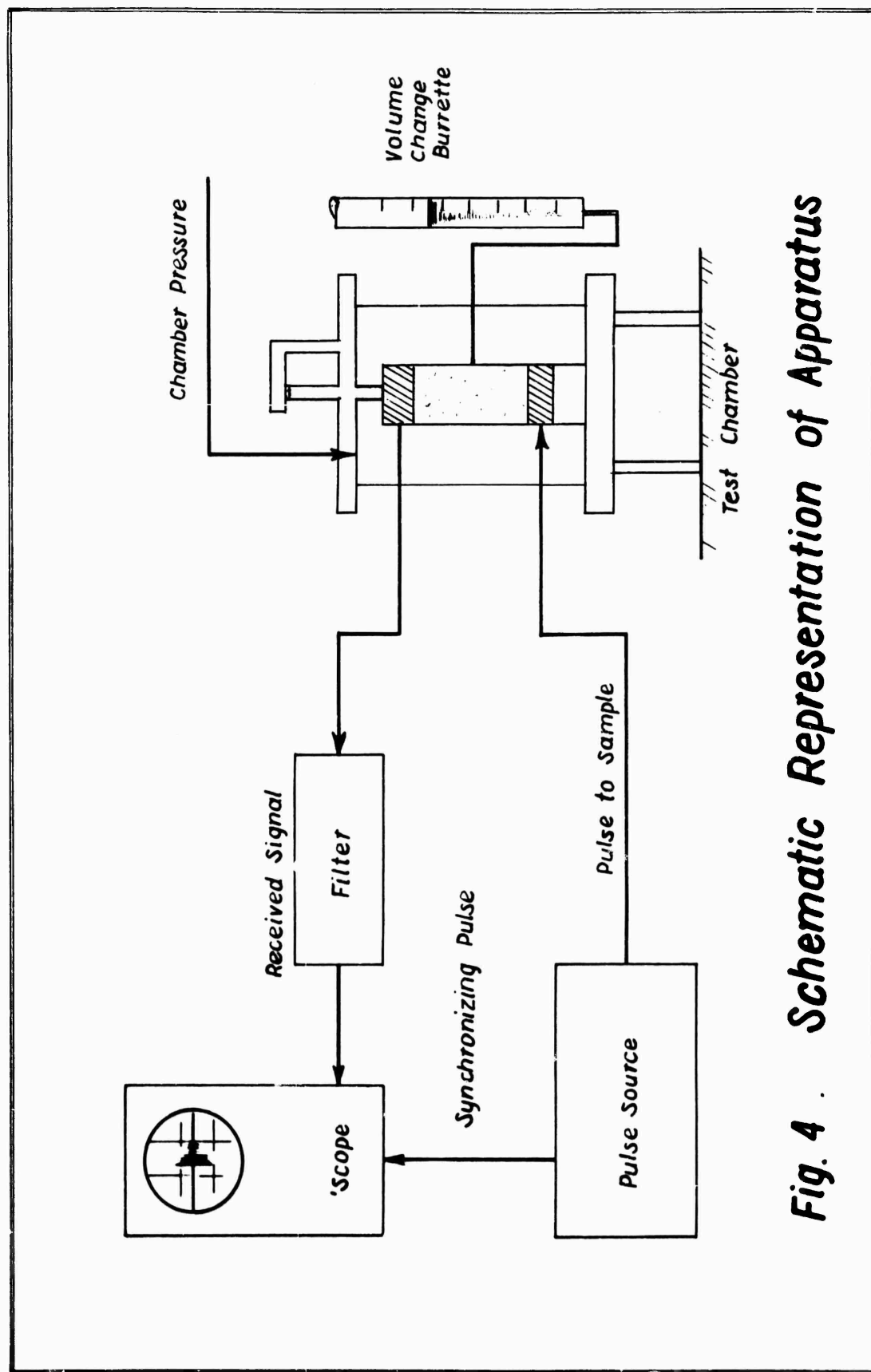
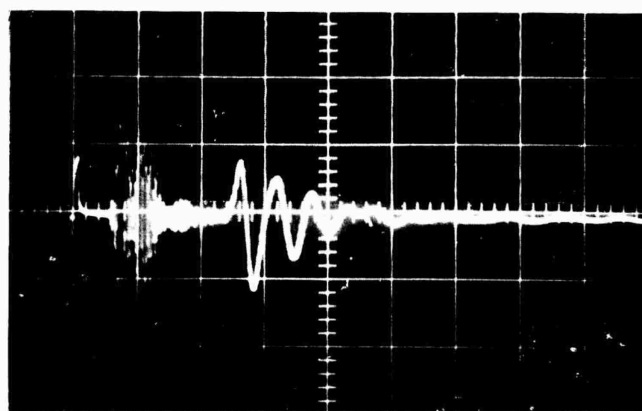


Fig. 4 . Schematic Representation of Apparatus



Shear Wave Arrival
 Dilatational Water Wave Arrival.
 Start of Trace: Time Zero.

Time History of Signal Received Through Boston Blue Clay

Consolidation Pressure	=	80 psi.
Consolidation Time	=	4,320 min.
Shear Wave Transit Time	=	248 μ seconds
Shear Wave Velocity	=	1,260 ft/sec.
Vertical Scale of Photo	=	5 mv./cm.
Horizontal Scale of Photo	=	100 μ sec. / cm.

Figure 5: Oscillograph of Received Signal

Ottawa Sand

$G_s = 2.67$

20 - 40 sieve

Well-rounded quartz sand

Boston Blue Clay

$G_s = 2.78$

L.L. = 33%

P.L. = 20%

P.I. = 13%

S.L. = 17%

36% minus 2μ ; Silty clay containing Illite and Chlorite

Kaolinite

$G_s = 2.62$

L.L. = 59%

P.L. = 28%

P.I. = 31%

53% minus 2μ

100% minus 60μ

Note: G_s = Specific Gravity; Atterberg Limits, L.L. = Liquid Limit, P.L. = Plastic Limit, S.L. = Shrinkage Limit, P.I. = Plasticity Index.

Figure 6: Soil Properties

SUMMARY OF TESTS PERFORMED

Test No.	Description of Material	Void Ratio	Loading Schedule and Remarks
0a	Ottawa Sand	dry	
0b	Ottawa Sand	sat.	
1	Ottawa Sand	dry	
2	Ottawa Sand	dry	
3	Ottawa Sand	dry	
5	Boston Blue Clay-D	sat.	
6	Boston Blue Clay-D	sat.	
7	Boston Blue Clay-D	sat.	
8	Boston Blue Clay-F	sat.	
9	Boston Blue Clay-F	sat.	
10	Kaolinite-F	sat.	
11	Kaolinite-HF	sat.	
12	Kaolinite-HF	sat.	
13	Kaolinite-F	sat.	
14	Kaolinite-HF	sat.	
15	Kaolinite-HF	sat.	
16	Kaolinite-D	par. sat.	
		dense .55*	cycled then loaded to 100 psi.
		loose .59*	cycled then loaded to 100 psi.
		dense .55*	hydrostatic and shear loadings
		loose .58*	many cycles hydrostatic load
		dense .55*	many cycles hydrostatic load
		.87 - .60+	consolidation
		.87 - .60+	consolidation
		.86 - .61+	consolidation
		.82 - .58+	consolidation
		.80 - .56+	consolidation, rebound, reload
		1.35 - 1.03+	consolidation, rebound
		1.54 - 1.15+	consolidation, rebound
		1.52 - 1.08+	consolidation
		1.34 - .93+	consolidation
		1.54 - 1.05+	consolidation, rebound
		1.54 - 1.07+	consolidation, rebound, reload
		.83 - .64+	consolidation, rebound, reload

Note: D = dispersed, F = flocculated, HF = highly flocculated
dry = air dry, sat. = saturated, par. sat. = partially saturated
* estimated values
+ range of values: see Section 2.5

Figure 7: Summary of Tests Performed

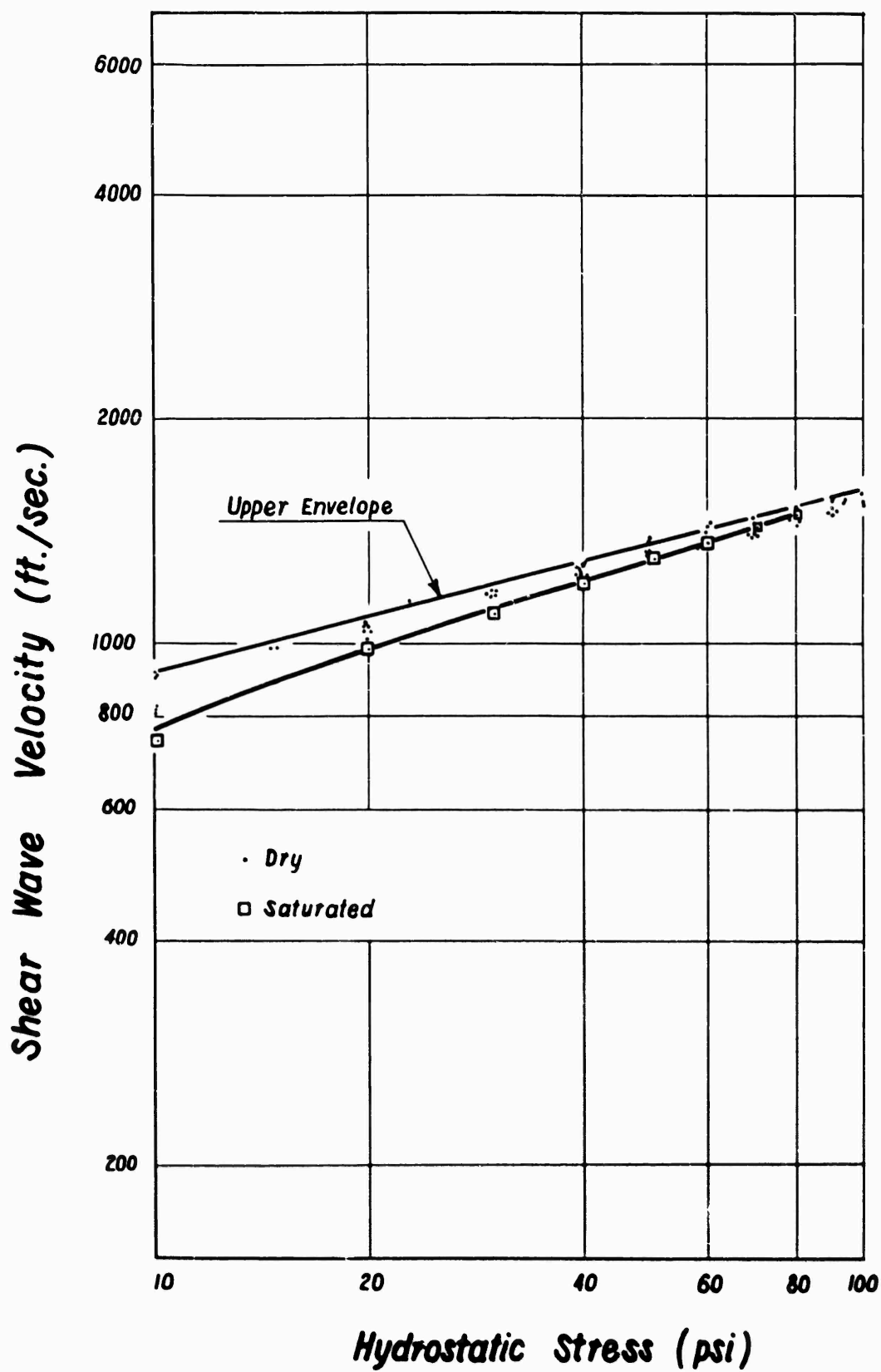
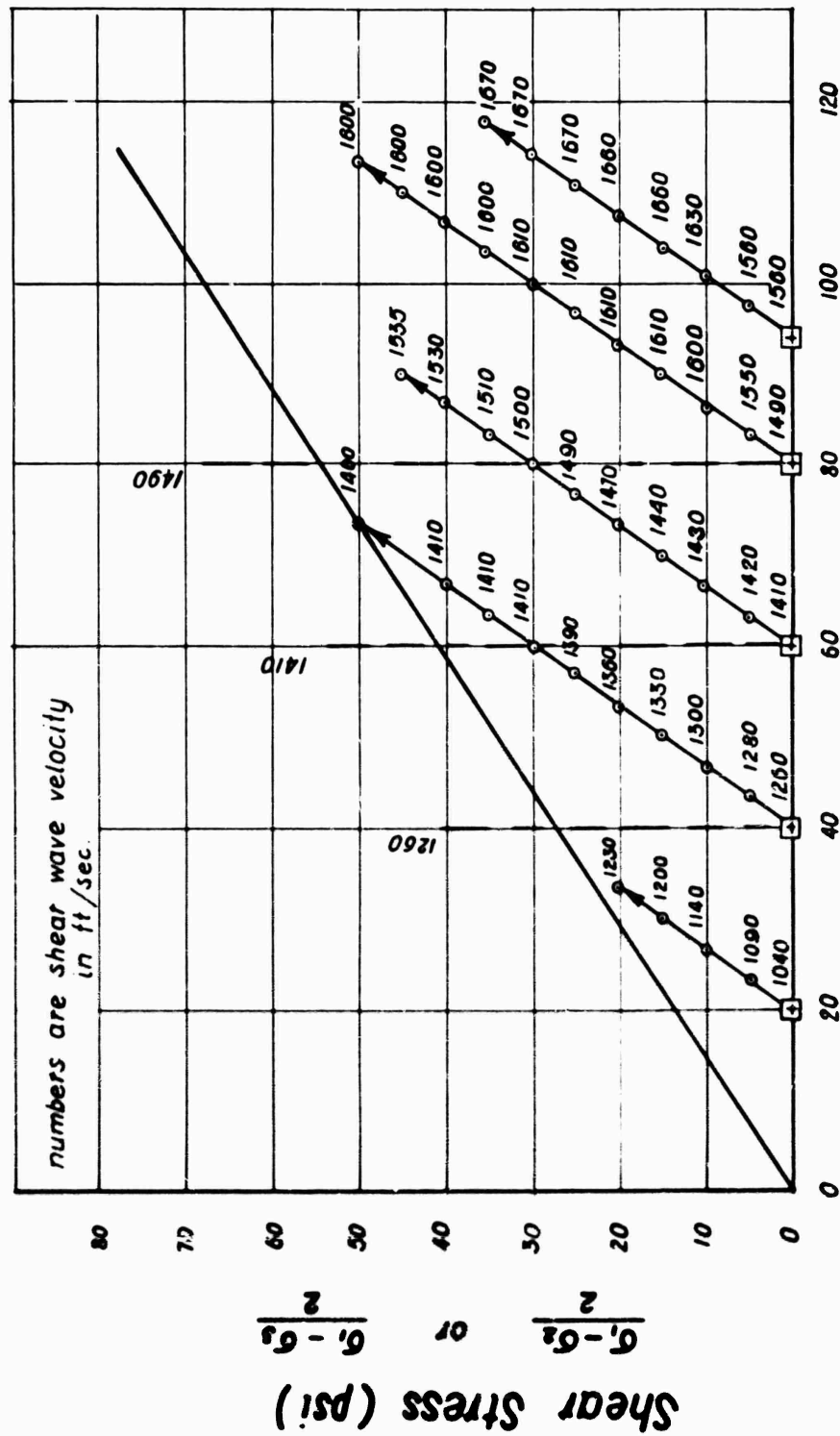


Fig. 8 Results of Hydrostatic Tests on Ottawa Sand



$$\frac{\sigma_1 + \sigma_2 + \sigma_3}{3}$$

Hydrostatic Stress (psi)

Fig. 9 Influence of Shear Stress on Shear Wave Velocity in Ottawa Sand

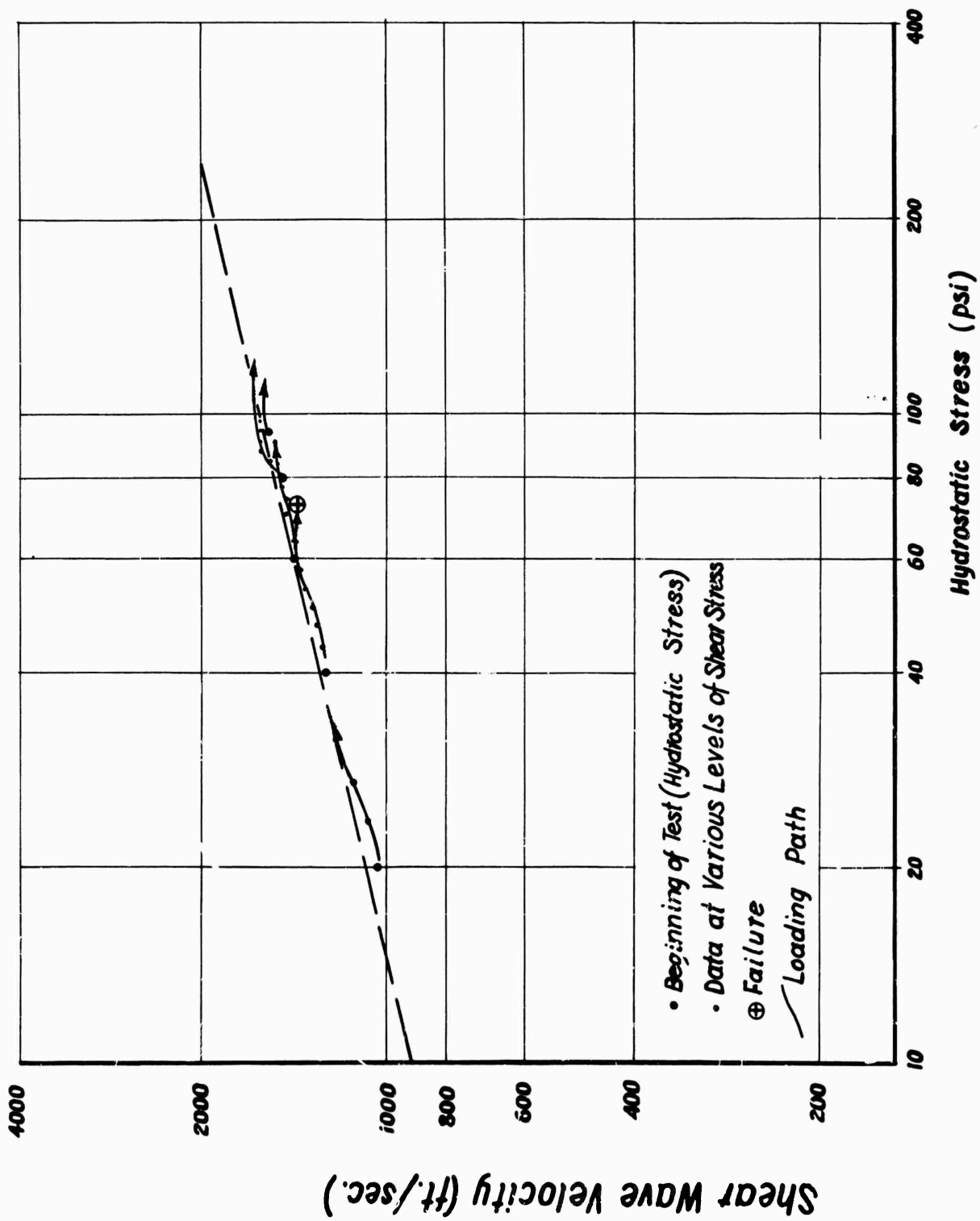
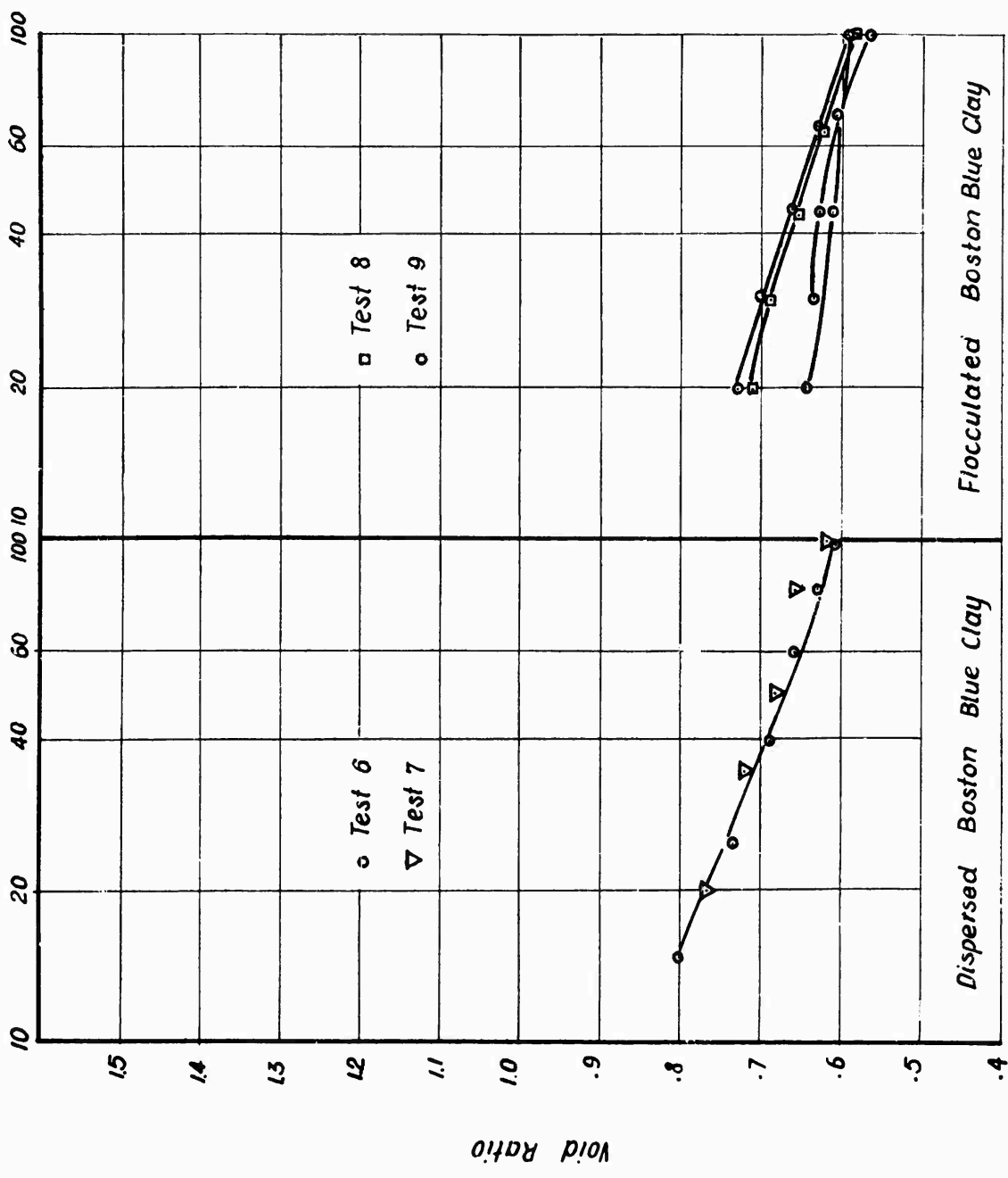


Fig. 10 Results of Shear Test Replotted from Fig. 9.



Hydrostatic Consolidation Stress (psi)

Fig. 11 Void Ratio - Pressure Curves for Boston Blue Clay

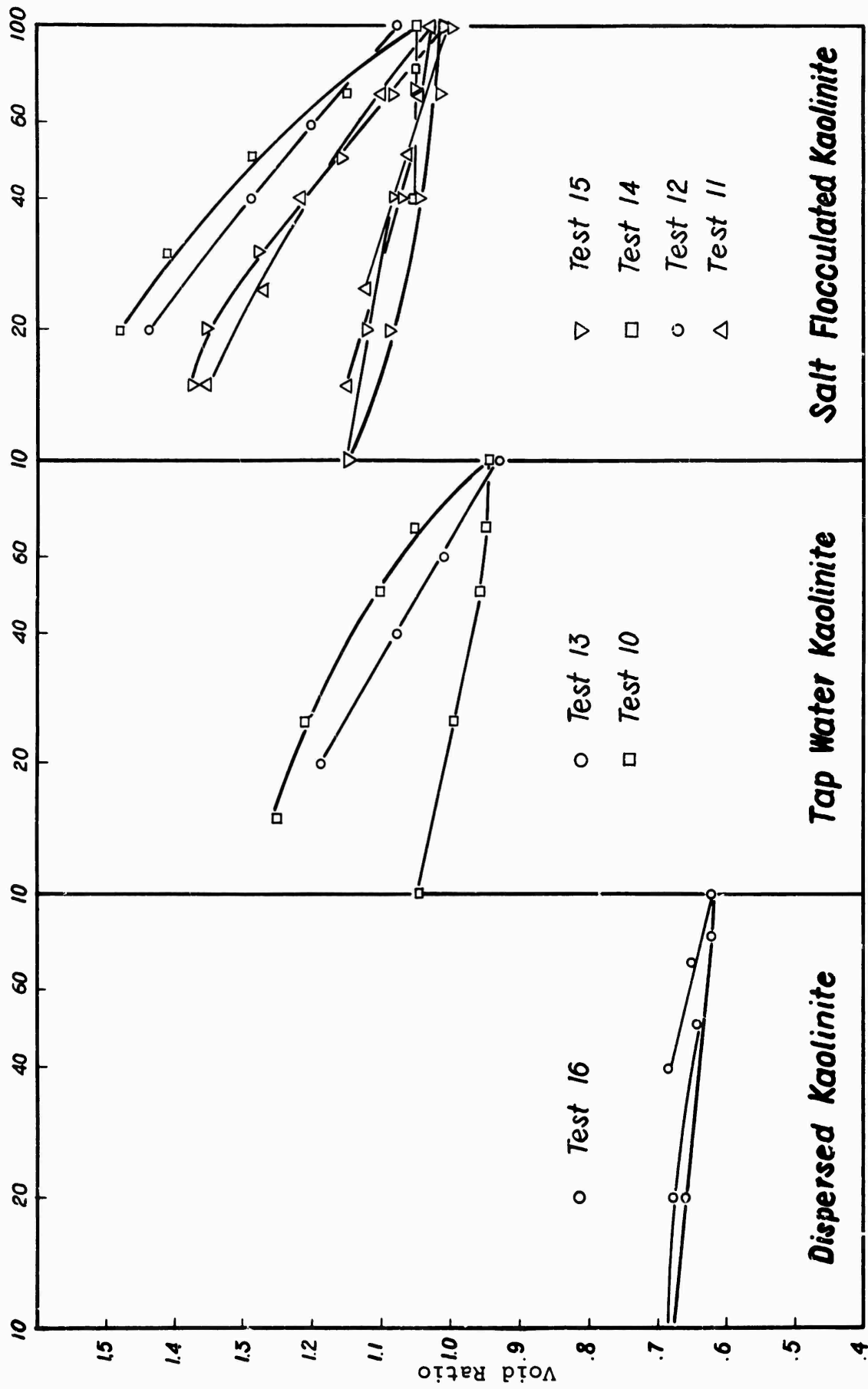


Fig. 12 Void Ratio Pressure Curves for Kaolinite Samples

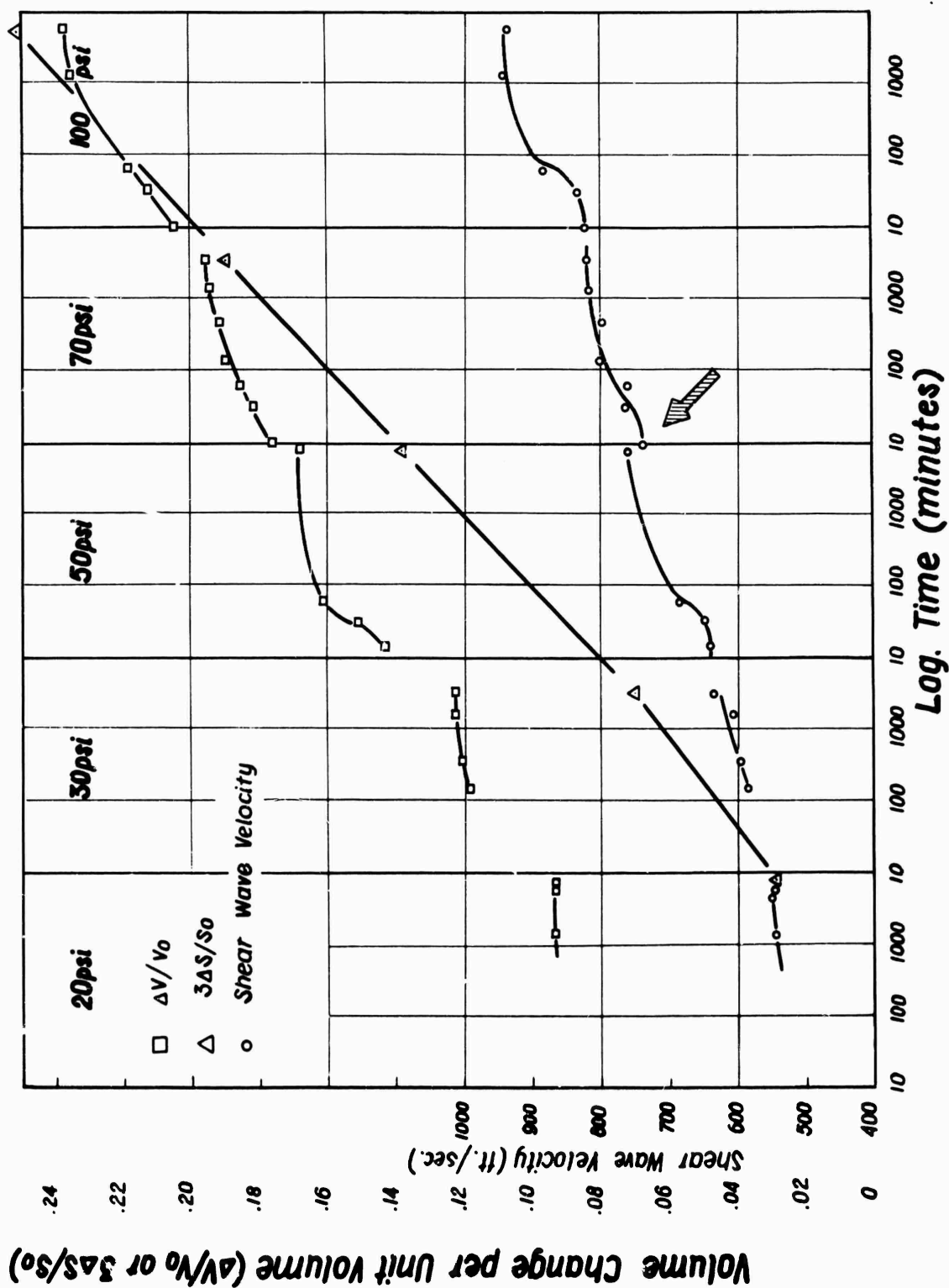


Fig.13 Time History of Hydrostatic Consolidation Tests on Clay, Load

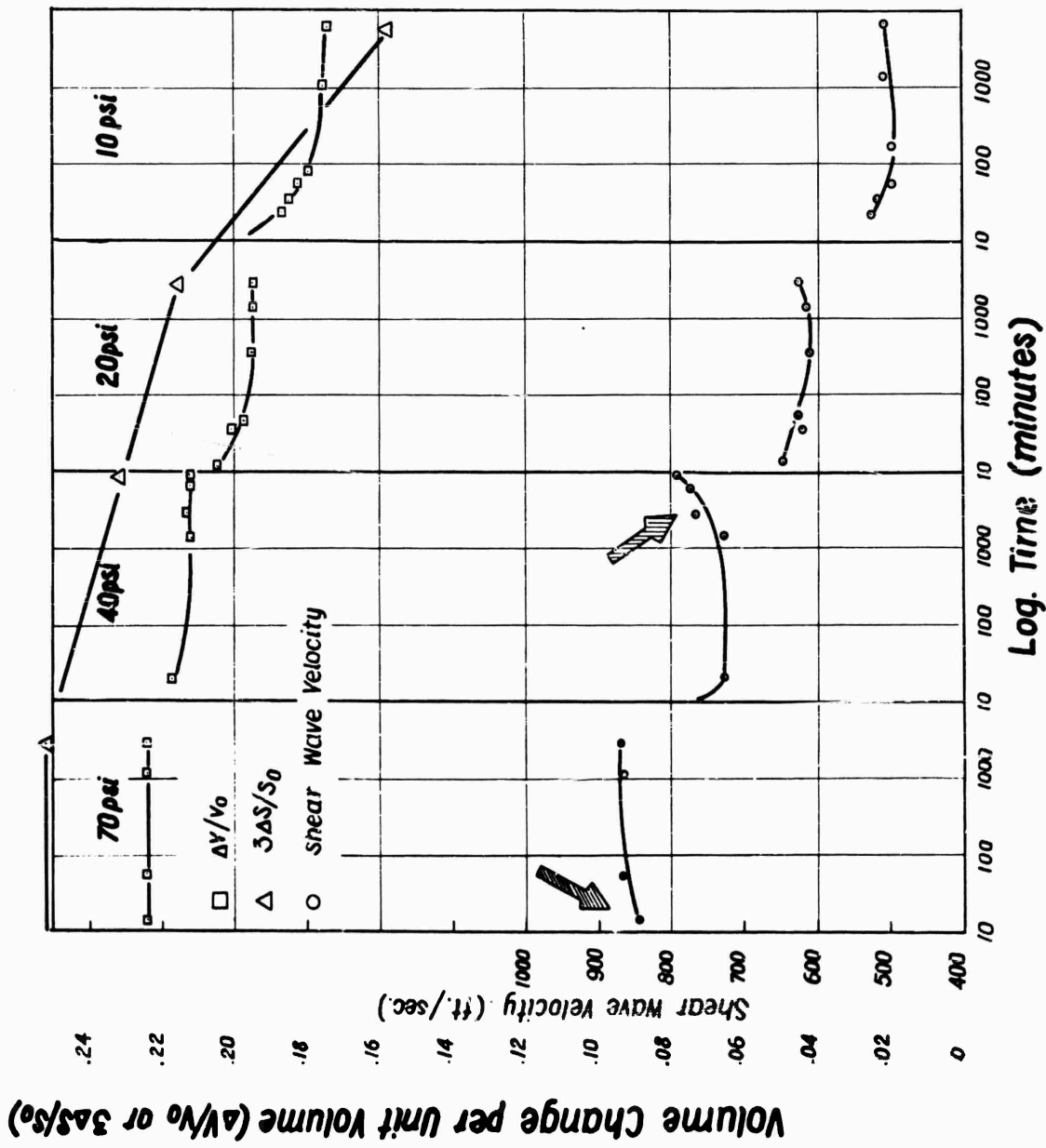


Fig. 14 Time History of Hydrostatic Consolidation Tests on Clay Unload

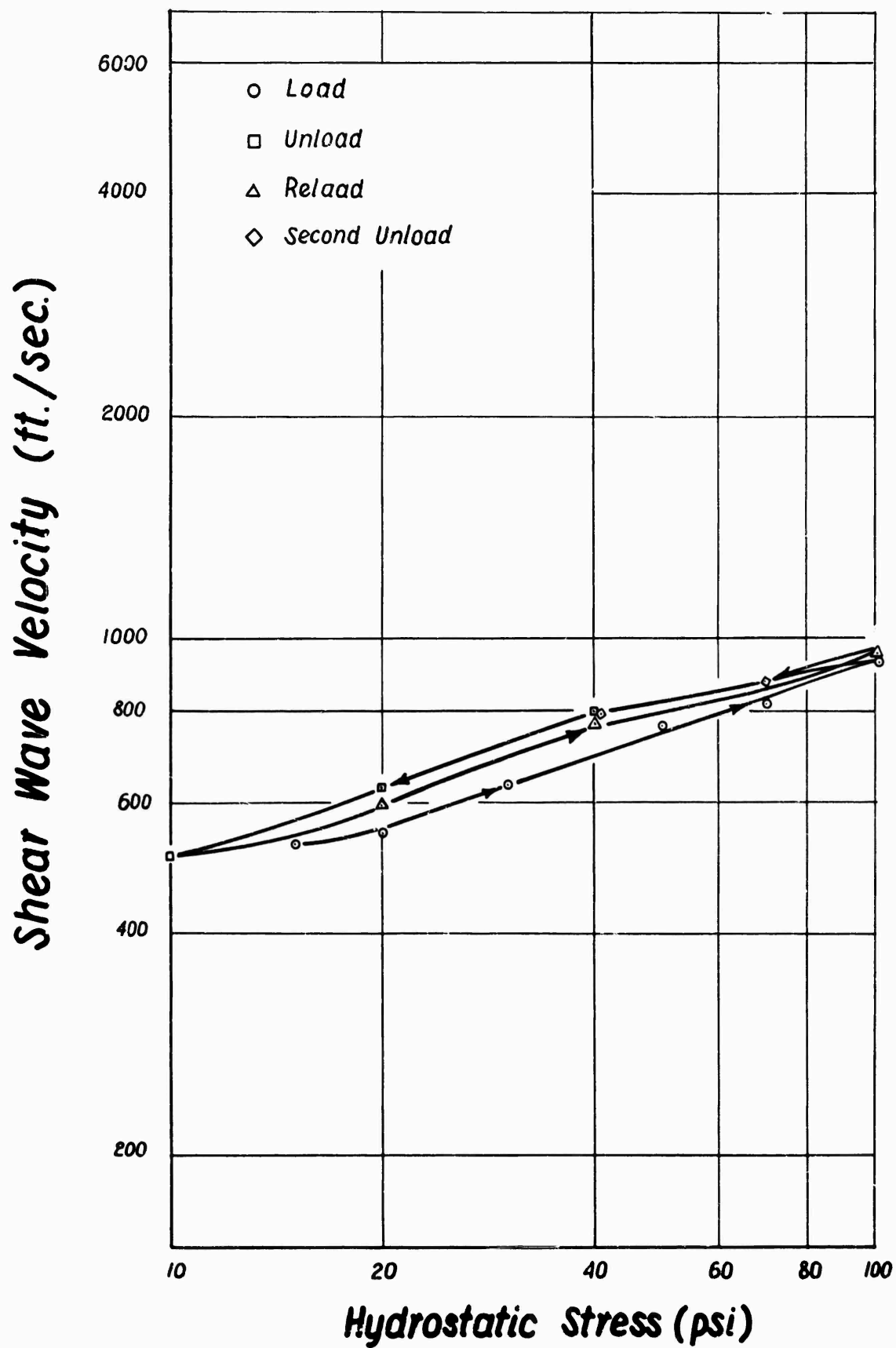


Fig. 15 Results of Hydrostatic Test on Clay

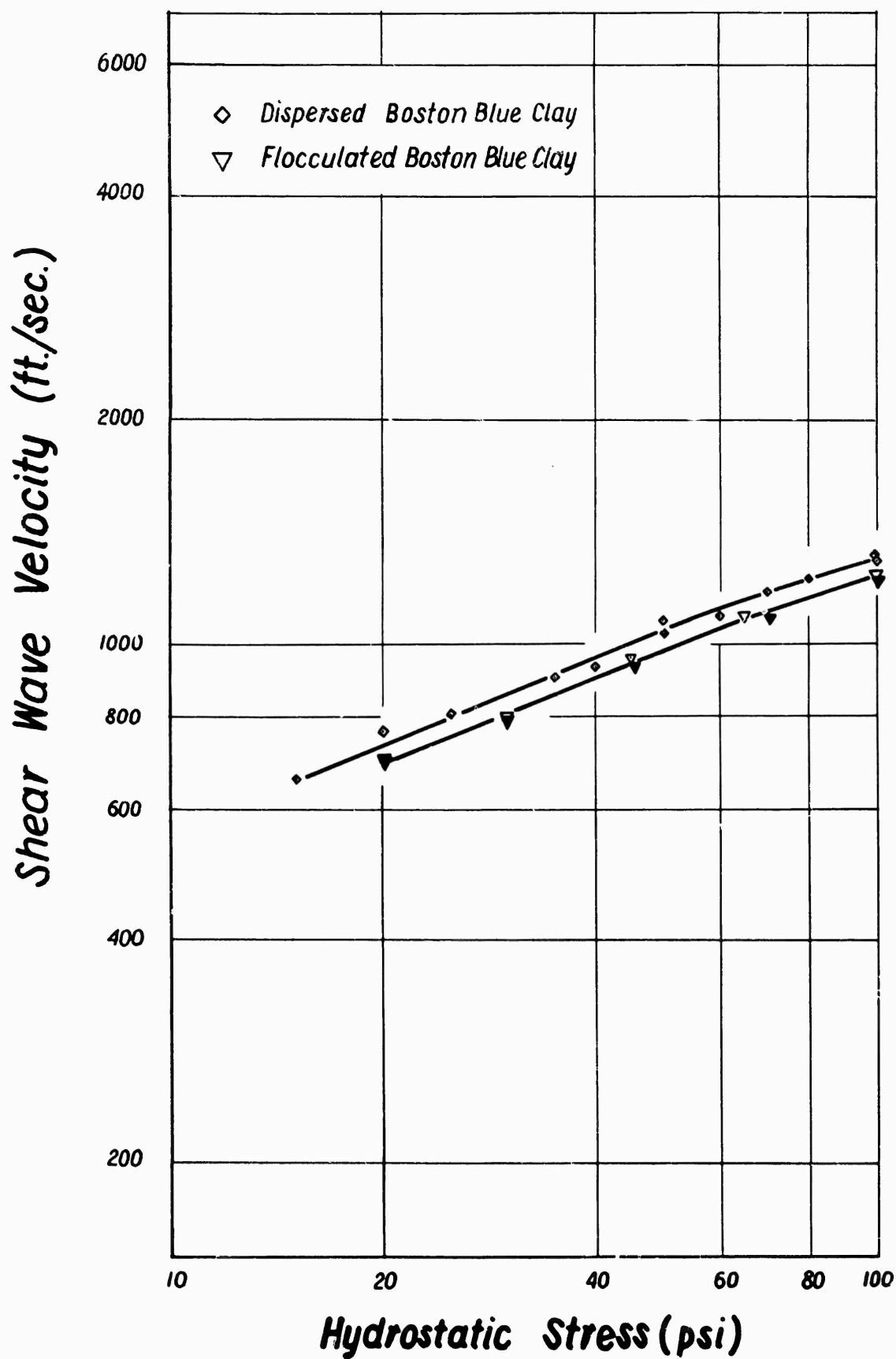


Fig. 16 Results from Loading Portions of Tests on Boston Blue Clay

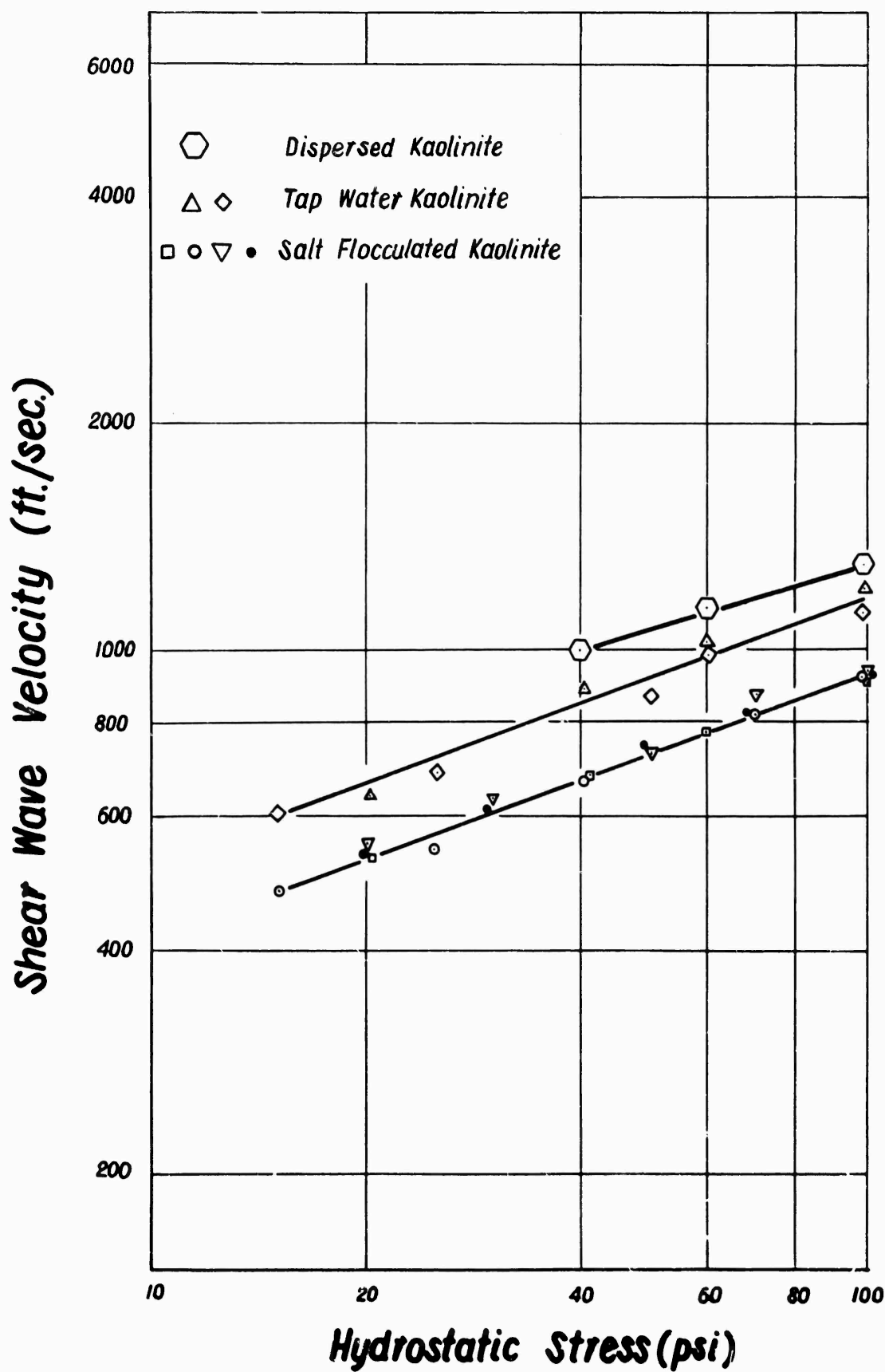


Fig. 17 Results from Loading Portions of Tests on Kaolinite

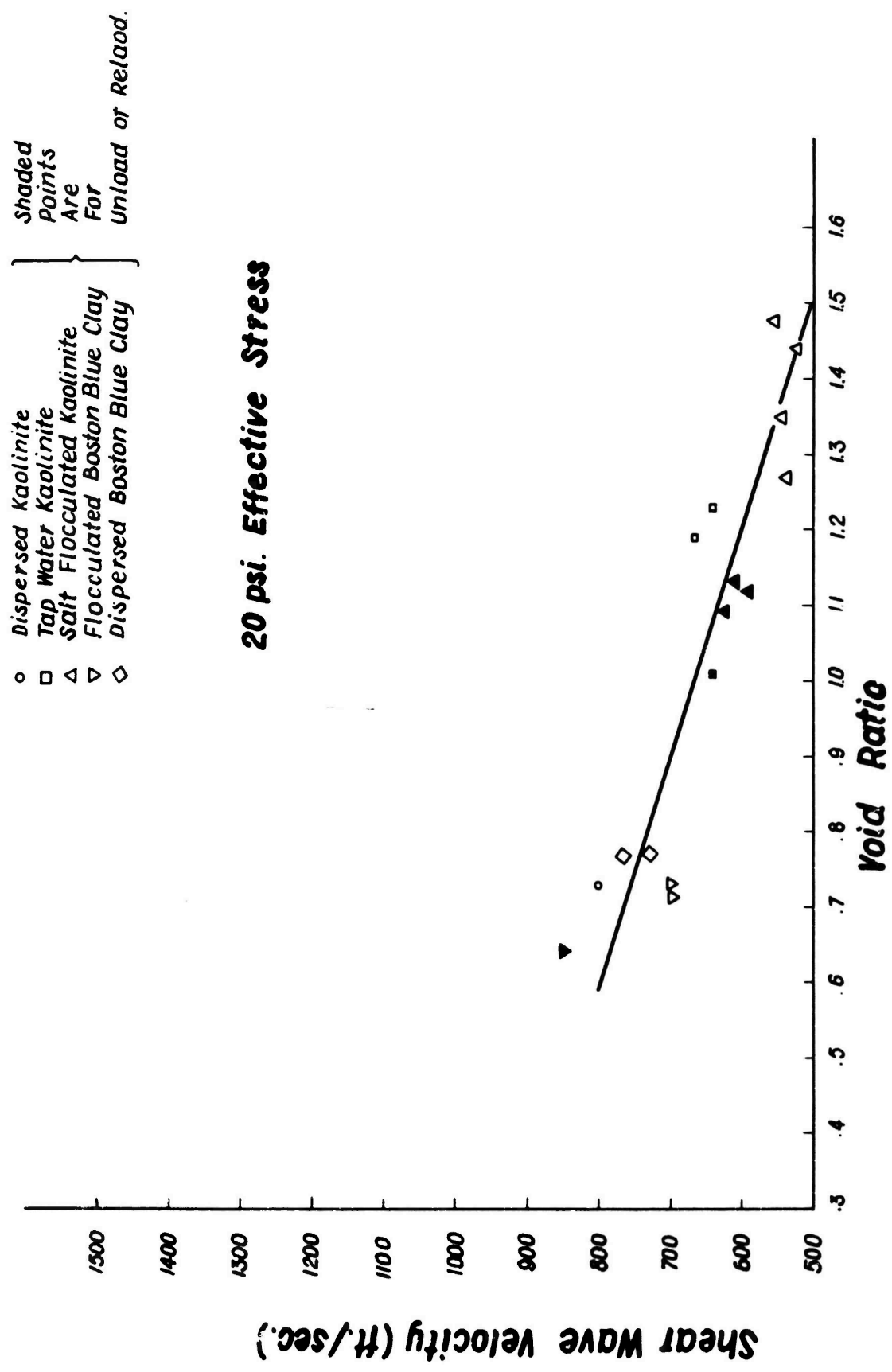


Fig. 12 Relationship Between Void Ratio and Shear Wave Velocity at 20 psi

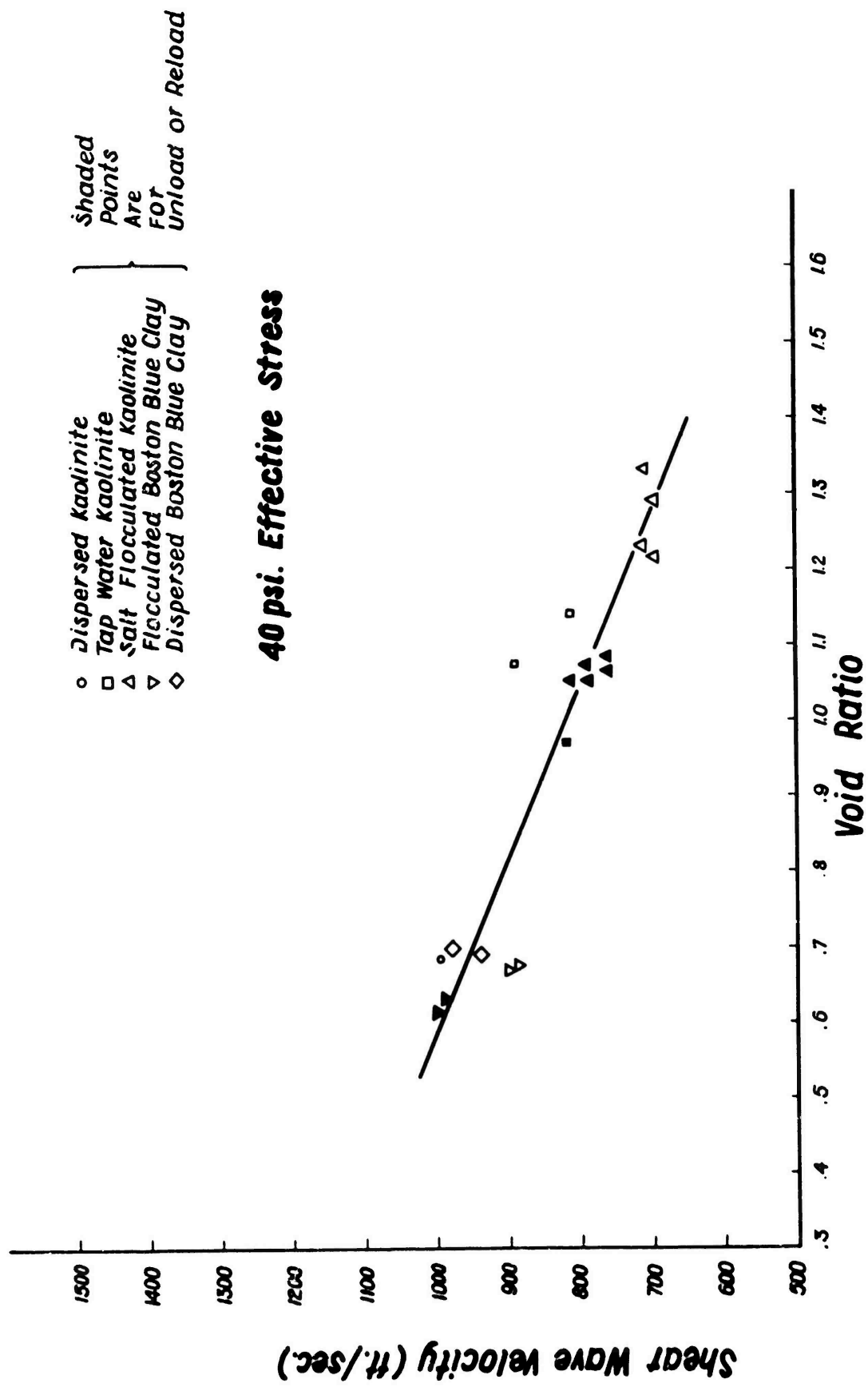


Fig.19 Relationship Between Void Ratio and Shear Wave Velocity At 40 psi

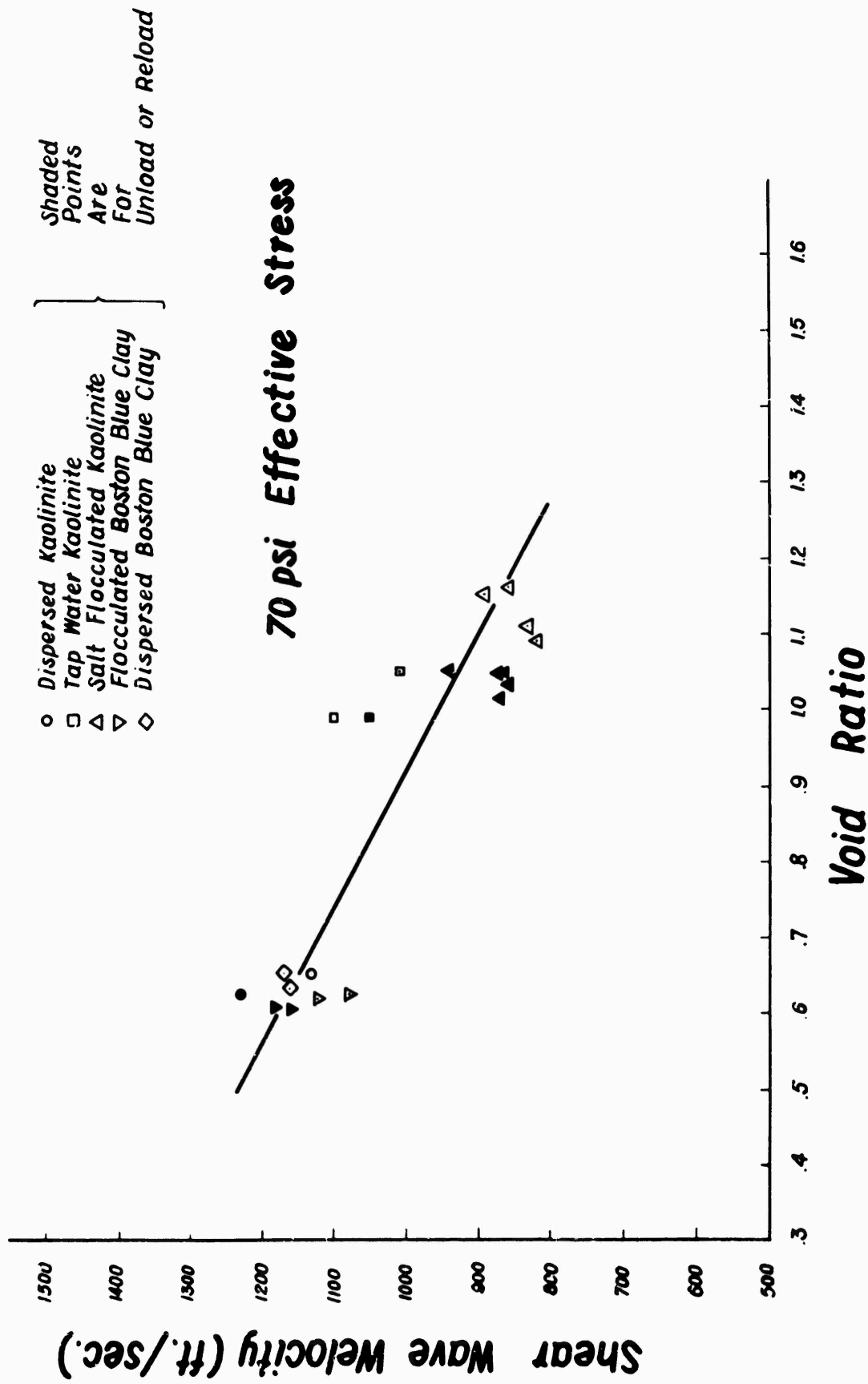


Fig. 20 Relationship Between Void Ratio and Shear Wave Velocity at 70 psi

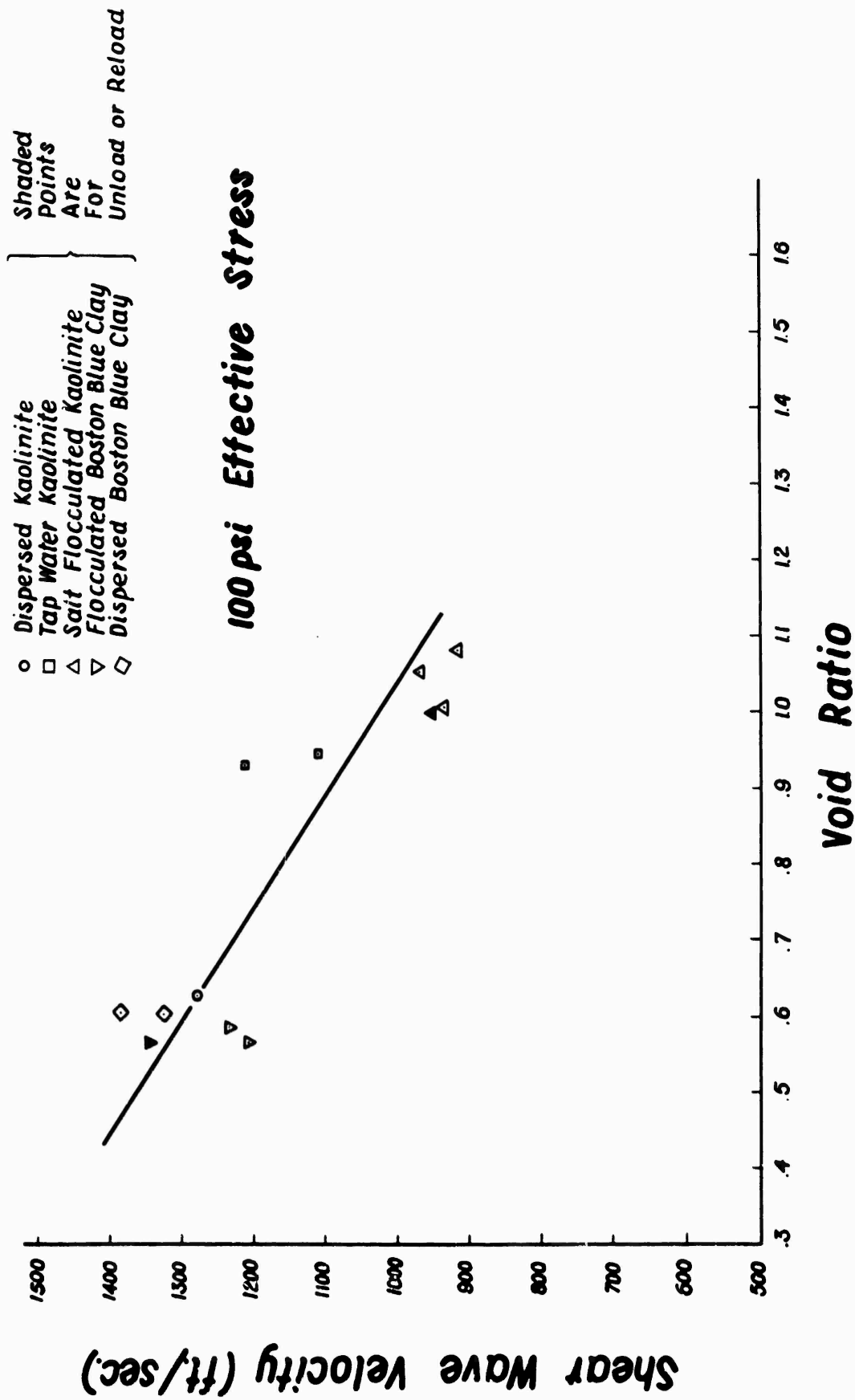


Fig. 21 Relationship Between Void Ratio and Shear Wave Velocity at 100 psi

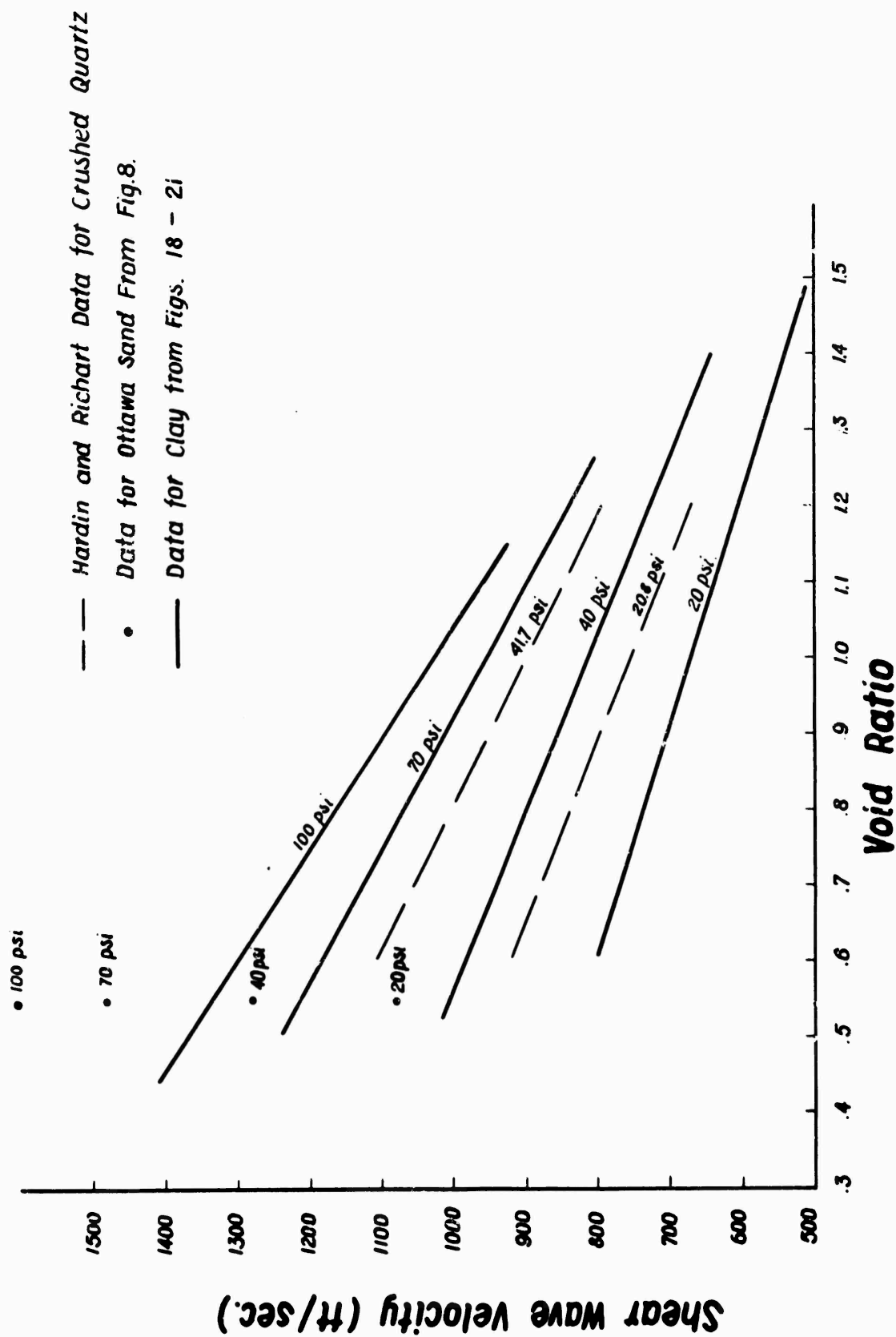


Fig.22 Comparison of Void Ratio-Shear Wave Velocity Curves for Various Materials

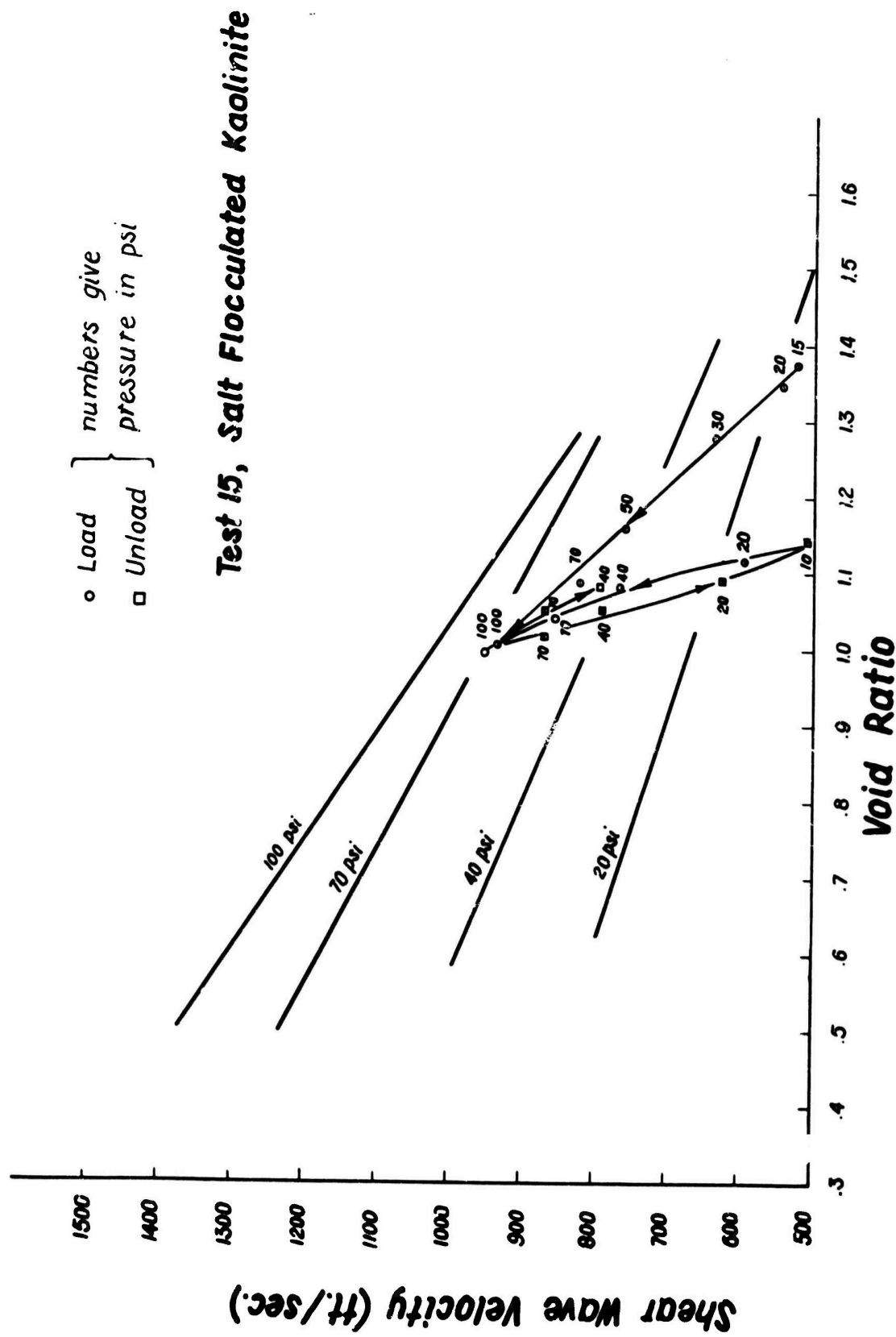


Fig. 23 Void Ratio Shear Wave Velocity Behavior During a Typical Test on Clay

APPENDIX A

RESULTS OF X-RAY DIFFRACTION STUDIES OF SOIL FABRIC

The fabric of the Kaolinite samples was studied by an X-ray diffraction method being developed at M.I.T. by Dr. R. T. Martin.* By measuring the intensity of X-rays diffracted from the fabric of a prepared clay surface, a dimensionless quantity, the Peak Ratio, can be assigned. Values of this ratio are proportional to the amount of particle alignment in the soil fabric. Highly oriented (dispersed) fabrics have characteristically large values of Peak Ratio (as high as 400). Completely random (flocculated) fabrics show much smaller values (near 1.0).

Samples are prepared for this studies by impregnating the wet clay with a moulten polyalcohol (Carbowax No. 6000) which replaces the water in the soil. Upon cooling, the Carbowax solidifies and permanently fixes the soil fabric; whereupon, the surface of the sample may be ground smooth for X-ray measurements without altering the structure being studied. The results of these studies are given below in terms of Peak Ratio.

	Remoulded and <u>Dispersed</u>	<u>Tap Water</u>	<u>Salt Flocculated</u>
	<u>Sample #16</u>	<u>Sample #13</u>	<u>Sample #12</u>
Before Hydrostatic Consolidation	1.3	9	14
After Hydrostatic Consolidation to 100 psi.	-	12	9

* See: Research on the Physical Properties of Marine Soils, Research Report R62-42, Soil Engineering Division Pub. No. 127, Department of Civil Engineering, Massachusetts Institute of Technology, November 1962, for a more detailed discussion of procedures.

The results indicate that what was presumed to be the most oriented fabric, that of No. 16, is the most random. The salt flocculated sample, presumed to be the most random, shows the most orientation. Further, this sample became less oriented, more flocculated, during hydrostatic consolidation; while the tap water sample tended to de-flocculate.

In general, these results are in conflict with the fabric information supplied by the void ratio-pressure curves of Figures 11 and 12 and with the fabric intended in the various modes of preparation.

More study is needed to resolve this conflict; but a partial explanation may lie in the fact that the X-ray diffraction method of determining soil fabric is extremely sensitive to every detail of the sample's history.